MASS. BRS1.2: Ev13

UMASS/AMHERST
312066 0269 7488 2

GOVERNMENT DOCUMENTS
COLLECTION

[illegible] 1994

University of Massachusetts
Depository Copy

AN EVALUATION OF THE SEISMIC DESIGN PROVISIONS FOR EXISTING BUILDINGS

PREPARED BY

BOSTON BUILDING CONSULTANTS

FOR THE

MASSACHUSETTS STATE BOARD OF BUILDING REGULATIONS AND STANDARDS SEISMIC ADVISORY COMMITTEE

FEBRUARY 17, 1994

943/517

This report has been developed as a result of code change proposals submitted to the Board of Building Regulations and Standards (BBRS) by the Seismic Advisory Committee.

Funding for the study and printing of this document was provided for by the Massachusetts Emergency Management Agency in its continuing effort to encourage earthquake preparedness and mitigation.

A. David Rodham
Director

John Smith
Earthquake Program Manager

Recycled Paper

Publication No. 17592 - 105 - 1000 - 7/94 - 2.96 - C.R.
Approved by: Philmore Anderson III, State Purchasing Agent

MEMBERS - BOARD OF BUILDING REGULATIONS AND STANDARDS

Prof. Kentaro Tsutsumi, Chairman
Registered Professional Engineer
(Structural)

Daniel O'Sullivan
Inspector of Buildings (City)

Thomas E. Donovan
Building Contractor of One
or Two Family Homes

Richard MacIntosh
Inspector of Buildings (Town)

William P. Kramer, P.E.
Registered Professional Engineer
(Mechanical)

Douglas Cole Smith, A.I.A.
Registered Architect

Stanley S. Shuman, P.E.
General Contractor of
Commercial/Industrial
Buildings

Thomas L. Rogers, Chief
Division of Inspections
Ex-Officio Member

Robert E. Banks
Building Trades

William Dolan
Head of Local Fire Dept.

F. James Kauffman
State Fire Marshal
Ex-Officio Member

STAFF

Thomas L. Rogers, Administrator
Robert A. Anderson
Thomas M. Riley
Brian Gore, Prof. Engineer
Ken Lucas

SEISMIC ADVISORY COMMITTEE

Norton S. Remmer, P.E.
Chairman
Consulting Engineers

Dr. John T. Christian, P.E.
Stone & Webster Engineering Corporation

David O. Knuttunen, P.E.
LeMessurier Consultants

Francis D. Leathers, P.E.
Geotechnical Engineers, Inc.

Maurice A. Reidy, Jr., P.E.
Maurice A. Reidy Engineers

Wayne Siladi, P.E.
Weidlinger and Associates

Professor Kentaro Tsutsumi, P.E.
BBRS Member

Kenneth B. Wiesner, P.E.
LeMessurier Consultants

Joseph J. Zona, P.E.
Simpson Gumpertz & Heger, Inc.

Nicholas Mariani, P.E.
Charles T. Main, Inc.

Dr. Cetin Soydemir
Haley & Aldrich, Inc.

LOADS ADVISORY COMMITTEE

Dr. Frank J. Heger, P.E.
Chairman
Simpson Gumpertz & Heger, Inc.

Rubin M. Zallen, P.E.
Consulting Engineer

Hans William Hagen, P.E.
Chairman
Lim Consultants

Arthur L. Brown, Jr., P.E.
Boston Building Consultants

Stanley Shuman, P.E.
BBRS Member

Michael Joliffe, P.E.
Zaldastani Associates

OVERVIEW

The "Seismic Design Provisions for Existing Buildings" was prepared by the Seismic Advisory Committee and recently adopted by the State Board of Building Regulations and Standards. It is intended to be incorporated into the Massachusetts State Building Code as Section 3208.

This report describes the study of five existing buildings that have had additions or alterations within the last twenty years designed by Boston Building Consultants. The evaluation of the five buildings forms parts 2 through 6 of the report. For each building the requirements of the code provisions applicable at the time are compared with the new provisions and any differences highlighted, including the associated costs for both design and construction.

The building studies also discuss the decision processes involved in the new provisions and note any ambiguities or difficulties encountered in applying them. These ambiguities or difficulties and other points of interest are elaborated upon in Part 7 of the report. Appendices A-E include sample calculations and illustrations to assist in understanding the report.

Part 1 of the report presents the conclusions drawn from the study, and includes a tabular summary of the results of the individual studies.

	Childrens Museum	Thayer Hall	Bixby Crossing	Lowell Dye House	N. Reading Library
Structurally attached addition?	No	No	Yes	Yes	Yes
% increase in weight % increase in area	- -	- -	25 **63 ***	50 32	**150 *** 70
Reduction of lateral load capacity?	No	**Yes ***	No	No	No
Original hazard index New hazard index	1 4	2 2	3 2	3 3,4,5	2 4
Occupancy increased >25%	Yes	No	Yes	Yes	Yes
Cost >50% assessed value?	Yes	Yes	Yes	Yes	Yes
Seismic hazard category	**3 ***	**2 ***	2	**3 ***	3
Original requirements	New const. Variance	None	Addition	No reduction	New const.
New requirements	New Const.	New const. Hazards	New const. Hazards	New const. Hazards	New Const.
Modified K allowed? Reduction Factor	Yes -	Yes -	Yes 93%	Yes -	No -
Comparison of new vs. original	Tie walls to floors better	Reinf. CMU corridor walls	Probably the same	Tie walls to floors	Same
Construction Cost $	6m	7m	7.5m	2m	3m
Design Fee $	40,000	50,000	60,000	20,000	25,000
Lateral retrofit Costs: Work carried out $ Additional work req. $	 750,000 -	 60,000 -	 150,000 -	 - 60,000	 20,000 -
% of construction cost	12	1	2	3	1
Additional design fees $	5,000	4,000	4,000	5,000	0
% increase	12	8	7	25	0

* Denotes governing factor

Part 1

CONCLUSIONS

1.1 EXISTING PROVISIONS

The requirements of the existing Building Code for the design of existing buildings are tied to a building's hazard index classification. This was developed from consideration of fire hazards, but is currently equally applicable to seismic design. In many cases a building's risk from fire and earthquake bear no relation to each other and there is little logical basis in applying these provisions to seismic design.

The fact that the existing provisions were written with fire hazards in mind has two other effects, when trying to apply them to seismic design: the requirements are frequently imprecise, and in many cases they are impractical.

There are currently no specific seismic design requirements in cases where the building's hazard index is not increased. If it is increased by one, then the existing code requires only that the building's lateral load resistance not be reduced. Increasing the hazard index by two or more requires the building to meet the code for new construction. There are problems with all three situations when applied to seismic design.

In the first case, with no increase in hazard index, it seems to be permissible to reduce the building's lateral load resistance, by removing walls, for example, which provide some lateral load resistance in a building that was not actually designed to resist lateral loads but nevertheless had some capacity to do so.

The requirement to not reduce a building's lateral load capacity can be, and often is, liberally interpreted. Consider the example of a brick masonry bearing wall building with wood floors not tied into the walls, a common building type in Massachusetts and perhaps the largest seismic risk. It can be argued with some conviction that the lateral load capacity of these buildings is governed by the connections of the floors to the walls. The lateral load capacity of the masonry walls acting in shear is far in excess of this, and therefore removing large amounts of masonry wall will not reduce the overall capacity which is still governed by the floor tie detail.

The existing requirement to meet the code for new construction has a different problem. It is impractical, or at best difficult to achieve. Section 1113.4 of the Building Code which covers seismic design is specifically limited to structures meeting certain ductility requirements set out in Section 1113.5. With specific detailing requirements for reinforced concrete, and the requirement for masonry to be reinforced, these provisions are hardly ever met for structures designed prior to the introduction of seismic design into the code in 1975. Section 1113.7 of the code does allow an alternative; for an engineer to determine that there is negligible risk to life safety in an earthquake with peak acceleration of 0.12g, based on dynamic analysis, comparison with similar structures that have withstood similar earthquakes, or other accepted procedures. A more common route is to obtain a variance, or assume one.

A similar problem occurs with additions. A vertical addition is usually very significant from a seismic point of view, but the only current code requirement is that it does not produce stresses in excess of those allowed by the code. No such allowable stresses are available for buildings not meeting the ductility requirements of Section 1113.5, and there is little guidance about how to deal with this situation.

In all of these possible situations, it can be seen that the aim of the code to gradually improve the seismic resistance of the building stock is frequently not successful. Much is left to the engineer's discretion, and while this is not in itself a bad thing, there is so much room for judgement that the special interests of building owners (not to spend money) can have significant influence.

1.2 APPLICABILITY OF NEW PROVISIONS

The new seismic design provisions of Section 3208 attempt to correct these problems by considering a building's seismic hazard separately from its fire hazard, and providing detailed guidelines for buildings that do not meet the ductility requirements of Section 1113.5. In our opinion the new provisions are successful in achieving this.

Although an initial reaction tends to be that the new provisions will require a lot more seismic retrofit work, our studies found this not to be true. The new provisions, being much more precise, remove the loopholes available for those wishing to avoid consideration of seismic resistance, but at the same time remove the usually unreasonable requirement to design buildings to meet the code for new construction. The actual requirements of the new provisions seem to lie somewhere between the most conservative and most liberal interpretations of the code as it stands. There will still be cases where considerable judgement is required and probably some where a variance will be needed, but these will be much fewer in number.

The major difference is that the design decision process is now based on factors specifically relevant to seismic design rather than fire hazards. The severity of the requirements depends on several considerations:

- Whether the cost of alterations exceeds 50% of the building's assessed value.
- Whether the occupancy in increased by more than 25%. This is only considered if the new occupancy is more than 100.

- Change of use of the building, based on hazard index. Previously the only deciding factor, this is now of much reduced importance and in effect only applies when a building changes to assembly type use.

- Buildings with additions have their own rules based on the scale relative to the existing building. Allowing structurally separate additions to be considered as separate buildings both simplifies the design work and encourages sound seismic design.

- Whatever other consideration apply, any alterations to elements of a lateral load system must be justified by analysis.

The new provisions still require the design to conform with the code for new construction in many cases, but usually allow a modified K value to be used for buildings that do not meet the ductility requirements. A lower level requirement is to correct special earthquake hazards - parapets, unrestrained masonry walls and connections of structural precast concrete elements.

It is worth noting that the only circumstance in which full compliance with the code for new construction including ductility is required is for a structurally attached addition larger (in weight or area) than the original building.

In the study of the five buildings we found that in three cases the seismic resistance work carried out under the contemporary code was of a similar nature and magnitude to that required by the new provisions. In a fourth, the new requirements had been met although technically they did not have to be. The fifth building would require more work under the new provisions, but only a relatively small amount.

It should be realized, however, that these buildings were not chosen at random, but specifically chosen to meet certain categories. It seems that probably the largest number of buildings will fall into a category where the only significant factor is the cost of alterations being more than half the assessed value. For these building the new provisions require the correction of special earthquake hazards as listed in Section 3208.6.3, whereas currently there are no seismic requirements.

Overall it appears that these provisions effectively achieve the aim of reducing the risk of loss of life by concentrating on specific areas of high risk resulting from seismic weakness or occupancy.

It is generally accepted that the largest earthquake liability in Massachusetts is from unreinforced masonry bearing wall buildings, and it seems that the major effect of the new provisions will be to require floors to be tied into walls in existing masonry buildings. Appendix F contains some typical details for this type of work that are included in the NEHRP Handbook for Seismic Rehabilitation of Existing Buildings (FEMA-172), which is readily available from the Federal Emergency Management Agency.

Section 3208.2 of the new provisions deals with the evaluation of existing buildings, and requires a substantial investigation and written report by the engineer. This appears to be a significant amount of extra work. The requirements are fairly specific but seem to allow scope for an engineer to decide when they are not necessary. Our suspicion is that the extent of these investigations will probably be reduced in most cases, and reach a level arrived at by consensus. The engineering peer review process and the education of building officials will probably be the determining factors here.

1.3 USABILITY AND CLARITY

Compared with the current code requirements the new provisions are more specific, more detailed and much longer. This is offset to some extent by their being self-contained and all together in the same place.

Our initial reaction was that the new provisions were complex, convoluted and difficult to understand in an abstract sense. This tends to be the nature of building codes in general, especially on first reading. We found, however, that with the familiarity obtained by studying the five buildings, they were not difficult to use, especially when read in the context of a specific building.

In practice, these provisions will probably be consulted at an early stage of design to establish what kinds of alterations to a building will be feasible. In that context they will be a little more difficult to use as an engineer will need to consider several possible situations at once, essentially following several simultaneous paths through the code. This seems to be something we will have to live with, as any simplification would compromise the logically thought out gradation of the requirements.

The major drawback to the clarity and ease of use is the frequent reference to Section 1113 and its subsections, some of which become unclear in the context of their link to Section 3208. Reference to Section 1113 seems to be unavoidable, but some clarification may be required, particularly in the case of forces on parts of structures. This is discussed further in Part 7 of this report.

On the whole, the new provisions are not difficult to use, though neither can they be described as simple. They seem generally to lead to a result which is clear in its meaning and commensurate with its intent.

1.4 COMPARISON OF COSTS

The study of the five buildings indicates that the new provisions will increase both design and construction costs, but by a small amount. Greater proportionate increases seem to apply to design, mainly due to the investigation, evaluation and report requirements.

In four of the five buildings studied, the total cost of lateral load retrofit work was 3% or less of the total construction cost, and in two of those buildings this was already required by the current code. The fifth building had 12% of its cost attributed to lateral load retrofit already required. This is probably close to the maximum that will be encountered as the building was rather weak seismically but the nature of the alterations caused some of the most severe requirements to be met, while the amount of other work on the building was fairly small.

As noted in the previous section, the buildings studied were not intended to necessarily be a representative sample. In trying to extrapolate the studies to form general conclusions, the wide range of interpretations of the current code makes if difficult to establish a baseline. Our suspicion is that a large number of buildings will now need seismic retrofit work that previously had no such work done but that it will be limited to correcting special earthquake hazards. This will probably add about 1% to the cost of construction in these cases. The figure of 1% is also the average additional cost of the new provisions over the existing in our five study buildings, and is probably a reasonable estimate for the average additional cost in general. Of course some buildings will need no seismic retrofit and some may need more. It appears likely that the only instances where the additional cost of the new provisions will be more than 5% are those where variances were used to avoid unreasonable requirements of the current code.

The percentage increase in design fees appears to be somewhat higher, but as the total numbers are smaller the overall economic effect will be less. Our studies indicate design fees increasing by an average of about 10%, but by up to 25% on smaller jobs. The total amount is usually around $5,000 and this obviously becomes very significant if the original fee is only $20,000. A significant part of the increase is attributed to the investigation and evaluation of the building and the likelihood that this will include some material testing. Perhaps the extent of investigation and testing will be less than we envision. It is also possible that a "typical" building would require less investigation than the buildings studied, which were intentionally chosen to be more complex examples.

1.5 AMBIGUITIES

Ambiguities, discrepancies or difficulties in using the new provisions are discussed in some depth in Part 7 of this report, and briefly summarized here.

Hazard index is of much less significant than previously, but still presents one small difficulty as a result of its link to fire hazard. This is the allocation of hazard index 8 to situations with high (fire) hazard, which is probably not applicable to seismic design.

Unreinforced brick masonry can now be considered part of the lateral load resisting system with a modified K value, but it is unclear how its allowable shear stress should be determined.

The tie force required to be transferred between masonry walls and floor diaphragms is not always clear, mainly because of apparently misleading wording of footnotes to Table 1113.2.

There are two minor ambiguities in the section regarding increases in weight or area: the need to use the larger figure not being clearly stated, and the question of whether or not to include unused attic floors in the calculations.

The correction of special seismic hazards seems to be considered as a less severe requirement than a full analysis using modified K values, but this does not always seem to be the case. It may be necessary to require the correction of special seismic hazards as well as the full analysis with a modified K value.

1.6 LIMITATIONS OF THIS REPORT

It should be stressed that the five buildings studied cannot be considered representative of all buildings to which alterations are made. The buildings were chosen in an attempt to test the new provisions in a series of interesting and challenging circumstances. Furthermore, the findings cannot even be considered typical for those circumstances as in each case there was a complex interplay of other parameters. Changes in use and occupancy, additions to buildings and modifications to existing lateral load resisting systems are frequently interlinked. The studies did show however, that the new provisions are comfortably able to deal with these complex combinations of parameters and produce reasonable results.

The approaches adopted to the design of the five buildings at the time represent the interpretation of one engineering firm. Other companies would undoubtedly have dealt with each situation differently given the wide range of possible interpretations of the current code. The basis on which comparisons are made to the new provisions may therefore be a little tenuous. From past experience, however, our belief is that our approach is neither at the conservative nor the liberal extreme but somewhere in between.

The study of foundations was considered beyond the scope of this report. As information may not be readily available, and foundations are only rarely a seismic problem, we made the assumption in each case that the lateral force can be transferred to the ground by some combination of friction and passive resistance of the soil. In each of the five cases studied the new provisions required investigation of the subsoils and the condition of existing foundations, and an evaluation of liquefaction potential. This would probably be carried out by a geotechnical engineer, and unless results from the original site investigation were available, would add to the project cost.

Part 2

THE CHILDRENS MUSEUM

2.1 BUILDING DESCRIPTION

A typical late 19th century New England mill type building of six stories, the Atlas Stores warehouse had brick masonry load bearing walls 20" thick at the bottom reducing to 15" higher up, and heavy timber floors.

The building is 370' by 70' in plan with transverse masonry bearing walls typically at 70' on center with two rows of 14" square wood columns between, spaced at 14'. Over the columns are 14"x16" wood girders, with 8"x14" wood joists at 3' on center over these. The floor itself is 3"x6" tongue-and-groove plank. The roof framing is similar but a little lighter, and columns and girders are smaller in the top two stories. Positive connections are at a minimum. Foundations are wood piles with stone caps. A floor plan and building section are included in Appendix A.

2.2 PROPOSED ALTERATIONS

The main alteration was to change the use of the building from low hazard storage to a museum, classified as an assembly place. Other minor alterations such as new door openings through walls were also planned.

2.3 CONTEMPORARY CODE REQUIREMENTS

The design work was done in 1977-8, shortly after the introduction of seismic requirements into the Massachusetts Building Code, but before the inclusion of Article 22 (now 32) in 1980.

In this Code a change of use to assembly use required a building to comply with the requirements for places of assembly thereafter erected. It also noted that the Board of Appeals may allow a variance.

2.4 METHODS OF COMPLIANCE

Two major obstacles prevented full compliance with the code for new construction: the masonry was unreinforced and the floors were not well tied into the walls. There were other less important problems such as the presence of lime mortar which was prohibited.

It would have been possible, but very difficult and costly, to add a thin reinforced concrete veneer tied to the brick walls, and a thin reinforced concrete floor topping over (or instead of) the existing floor, and tie them together.

However, the most feasible course of action was to identify the building's seismic points of weakness, and to greatly enhance its resistance. This was accepted by the building official as a variance. Plywood sheathing was added over the floor to allow it to act as a diaphragm, and the floor was tied into the walls with threaded rods grouted in place and provided with plate washers. In addition, existing openings in the walls were filled solid with masonry if they were no longer required, to compensate for the new openings being cut.

Without any quantitative analysis, this was considered as the most efficient and cost effective way to improve the building's seismic resistance.

2.5 REQUIREMENTS OF NEW SECTION 3208

There being no addition, and no reduction in capacity (assuming new openings are offset by infilled ones) the change in use from hazard index 1 to 4 and increase in occupancy by more than 25% and to more than 100 puts the building into seismic hazard category 3 from Table 3208.1.

This requires full compliance with Section 1113, but allows for the use of K = 4.0 in lieu of the detailing requirements (for masonry to be reinforced). The required strength of the wall to floor connection will be determined from 1113.4, the minimum of 100 or 200 pounds per foot does not apply.

2.6 POSSIBLE METHODS OF COMPLIANCE

Using K = 4, the total base shear V is 3,065 kips in the long direction, giving a shear stress of 25 psi in the brick masonry walls. In the short direction V is 2,316 kips giving a shear stress of 28 psi. These are within the allowable limit of 37 psi calculated from BIA-69, but this is discussed in more detail in Part 7.

Calculation of the force required to be transferred at the floor to wall connection is far from simple. Using the values of C_p from Table 1113.2 gives a maximum value of 150 pounds per foot, which is in line with figures given elsewhere in the Code for other circumstances. Difficulties arise when referring to notes 3 and 4 to this table, which are discussed in Part 7. Complying with these notes gives forces of 500 pounds per foot at the higher levels.

Either way, the qualitative solution provided in the late nineteen seventies probably falls short of the new requirements. More continuous tying of the wall to the floor would be required. Providing for 500 pounds per foot of force transfer could be achieved by adding steel straps fixed to the floor framing and attached to threaded rods passing through the walls with plate washers on the outside. These would be spaced at about 8' intervals, with solid blocking between the floor joists for nailing of the straps. The typical details in Appendix F would be applicable.

2.7 COST ESTIMATES

Because of the nature of the building, with large open spaces, overall renovation costs are not high, and the total project cost was about $6 million using present day values. Brick masonry bearing walls and wood floors poorly tied in is, however, one of the worst types of building from a seismic viewpoint, and consequently the structural strengthening costs are relatively high, around $750,000, which is over 12% of the total. This cost would be very similar to meet the requirements of the new provisions as for the empirical approach adopted as a variance in 1978. It is worth noting that the combination of factors for this building probably comes close to being a "worst case scenario" for the proportionate cost of seismic reinforcement.

Structural design fees of around $40,000 would probably not be affected, but an additional cost of around $5,000 could be expected, to carry out more detailed investigation and testing. This could be reduced significantly if masonry strength could be ascertained without any physical testing.

2.8 COMMENTS AND DISCUSSION

Section 3208 gives a clearly defined method for dealing with this building, and produces requirements which are readily achievable by a slightly modified version of the system that required a variance to meet the Code at the time.

A more extensive investigation of the existing structure would be required, which would probably include testing the strength of bricks and mortar, and better determination of the methods used to connect the beams into the walls.

Specific problems encountered are discussed in Part 7; how to determine the allowable shear stress of old masonry, and the notes to Table 1113.2.

Part 3

THAYER HALL

3.1 BUILDING DESCRIPTION

One of the Victorian brick buildings surrounding the quadrangle at Harvard Yard, Thayer Hall has been a student dormitory since its construction in 1869.

In plan it is 210' by 45', consisting of a central 60' long block of 5 floors and symmetrical wings of 4 floors. In addition there was an occupied basement, and an unused attic floor throughout. The central block had a stairwell and hallway laterally across the center of the building, the two end blocks each had a longitudinal corridor over about half of their length. The three blocks were not accessible from one another, but were structurally integral.

The structure is load bearing brick walls and wood joist floors. There are sixteen interior walls across the building and eight chimneys running the full height of the building and through the roof. The direction of the joist span varies, some bear on the exterior walls, some on the interior cross walls. The slate roof is supported by heavy timber posts and purlins with wood rafters. Appendix B includes a typical floor plan identifying new walls added and old ones removed.

3.2 PROPOSED ALTERATIONS

An extensive renovation was designed in early 1993, the notable features of which were to introduce habitable space in the attics (6th floor in the center, 5th in the north and south wings) and to improve access and egress by cutting a continuous corridor running north-south through the building. This included the removal of four of the chimneys, and several pieces of load bearing wall around them.

3.3 CONTEMPORARY CODE REQUIREMENTS

As there was to be no change in use, and no structural additions, there were no code requirements for lateral load, indeed a very literal approach could conclude that the code did not even prohibit the removal of lateral load resisting elements in this case.

3.4 METHODS OF COMPLIANCE

At the time of design, it was expected that Section 3208 may be introduced to the code prior to construction of the building. In any case, removal of walls that contributed to the building's lateral load resistance was not considered acceptable without some form of check.

The draft Section 3208 was used as a guideline and showed that the walls remaining were adequate in the east-west direction. In the north-south direction the lateral load resistance was much lower, but new corridor walls were to be constructed, and it was reasonable to use reinforced masonry for these. This would give a very good system to resist north-south lateral loads. Without lateral load considerations, it would have been possible to use metal stud walls instead and add steel framing to support the floors, but the savings in time and cost were not expected to be very high.

3.5 REQUIREMENTS OF NEW SECTION 3208

In applying Section 3208 more rigorously, some interesting points arise. Making the attic floor habitable could be considered to increase the floor area by 20%, but it is not clear whether or not this is the intention. It certainly increased occupancy by 20% but that is below the threshold in Table 3208.1.

The cost of alterations would probably be more than 50% of the assessed value of the building. The value is uncertain and for a building such as this the figure may be open to manipulation. In addition, note 3 to Table 3208.1 allows the deduction of work to meet ADA and Access Board requirements; the corridor could possibly be considered as such which could conceivably keep the cost below the 50% threshold.

The seismic hazard category would be 1 or 2 depending on whether the cost was below or above 50% of the value. The only difference is the investigation of special earthquake hazards - connection of masonry walls to floor and roof diaphragm, and parapets (should the remaining chimneys be included ? - probably not).

All of this is somewhat academic as paragraph 3208.3.5 governs because there is a reduction in overall capacity. The building needs to meet the requirements of Section 1113, with K = 4.0 in lieu of the detailing requirements of 1113.5.

3.6 POSSIBLE METHODS OF COMPLIANCE

As was the intention, the building as designed appears to meet the requirements of Section 3208. Although the exterior walls are not tied into the floors, they are reasonably well tied into intersecting lateral walls at 15' intervals so they should be stable and allow the transfer of shear forces. Connections at interior walls are more positive because of the presence of floor on both sides, and the new corridor walls and slab being well tied into existing floors and walls should ensure their stability.

3.7 COST ESTIMATES

Out of a total project construction cost of $7 million, only about $60,000 could be attributed to meeting in advance the requirements of Section 3208. This is the estimated premium for reinforced masonry walls over metal studs and steel beams installed in tight working conditions.

The structural engineering fee of around $50,000 could be considered to include about $3,000 for seismic analysis. To meet the strict requirements of Section 3208 for investigation, testing and reports would probably add another $4,000 or so to the costs.

3.8 COMMENTS AND DISCUSSION

Section 3208 imposes more onerous requirements on this building than those in effect at the time, but those requirements do not seem unreasonable. The introduction of Section 3208 in this case makes the analysis process very clear. Previously it may well have been possible to weaken the building during its modification without any verification of its lateral load capacity, and any analysis attempted would be totally a matter of judgement.

Part 4

BIXBY CROSSING, HAVERHILL

4.1 BUILDING DESCRIPTION

This is a 160' by 60' building of eight stories, plus a basement which is above ground on one side of the sloping site alongside the Merrimack River in the center of Haverhill. It was constructed around 1930 as a shoe factory; its last use before being abandoned was for a box company.

The structure is a reinforced concrete flat slab system with an 8" thick slab and 4" deep drop panels at interior columns. Columns are spaced at 20' in both directions. Interior columns are round with flared capitals, diameters range from 28" in the basement to 16" at the roof, reducing in size every two floors. The reinforcement for these columns includes spiral ties. Exterior columns are rectangular, 16" x 32", with a 12" x 24" spandrel beam turned up from the edge of the slab. Except for a 2' high brick wall on top of the beam, the remainder of the exterior was glass. A building section and typical floor plan are included in Appendix C.

The foundation system consists of reinforced concrete piles at the river end, spread footings at the other end. The interior of the building contained no substantial walls, lateral load resistance was provided by frame action of the concrete.

4.2 PROPOSED ALTERATIONS

The 1988 remodeling to residential apartments extended the building vertically by four stories, and added a 12' wide by 100' long strip on one side of the building for its full height.

The additions utilized light steel framing, with a 2 1/2" thick concrete floor slab on steel bar joists. The main steel frame matched the column layout of the concrete structure. Exterior walls were metal stud backed E.I.F.S. panels.

4.3 CONTEMPORARY CODE REQUIREMENTS

As the hazard index was reduced from 3 to 2, the only requirement for the existing building was that the addition impose no loads causing stresses exceeding those permitted by the code for new construction (Section 3200.3 item 5)

4.4 METHODS OF COMPLIANCE

A fairly detailed analysis of the existing concrete columns was carried out to confirm their capacity to carry additional gravity loads. No structural drawings were available, so a field investigation was carried out to determine the steel reinforcing of the lower columns, and typical concrete compressive strengths.

Steel K-braces, designed to act compositely with the concrete frames, were added to sustain wind loads in the lateral direction of the building, and the composite system was checked using a computer stiffness analysis. A similar analysis of frames in the longitudinal direction confirmed the capacity of the existing columns to sustain the increased wind load by frame action. Steel moment frames in the top four floors meet all of the requirements for new construction.

It seems that no analysis of seismic loads was performed for the existing building. It is clear that wind loads govern in the lateral direction, but probable that seismic loads govern in the longitudinal direction. Rightly or wrongly, the requirement of Section 3200.3 item 5 was considered not to apply to seismic loads. If it were to apply, it is not clear exactly how it could be applied. The reference to causing "stresses exceeding those permissible" seems to imply no need to meet the design requirements of Section 1113.5, but this rules out the use of a K value from Table 1113.1. This apparently leaves the only option to be to meet Section 1113.7, requiring dynamic analysis, comparison with similar buildings that have withstood actual earthquakes, or "other accepted procedures" to determine that there is negligible risk to life safety in an earthquake with a peak acceleration of 0.12 g.

4.5 REQUIREMENTS OF NEW SECTION 3208

As the hazard index does not increase, the seismic hazard category from Table 3208.1 is 2 as a result of the occupancy increase and cost of alterations. This requires only that there is no reduction in lateral load and that special earthquake hazards be corrected; in fact there are none in this building.

The requirements of Section 3208.4 for additions are more severe. The addition is not structurally separated, the increase in weight is 25% and in area 63%. Assuming the larger value governs (see discussion in Part 7) this requires a modification factor of 93% (from Figure 3208.1) of the base earthquake force, with a K factor of 1.5 (Table 3208.2) for a reinforced concrete frame not meeting the design requirements of Section 1113.5. Although the columns have suitable spiral tie reinforcement, there are several other requirements of Section 1113.5.1 that are almost certainly not met, such as the reinforcement needing to meet ASTM A615, and the reinforcement requirements for flexural members in Section 1113.5.1.2 applied to the slab/beam elements. The new requirements in Section 3208.2 for evaluation of the building would determine whether this is in fact a correct assumption. This evaluation would have to be much more extensive than that carried out in 1987, and include detailed investigation of the slab reinforcement.

Force modification factors from Table 3208.3 should be used, but without detailed investigation of the slab reinforcement, it is uncertain which ones would apply. Note 2 from Table 3208.2 also applies; a two-stage process should be used with the flexible steel building analyzed as on a rigid base with K = 0.67 and its base force applied to the top of the lower concrete building with K = 1.5.

4.6 POSSIBLE METHODS OF COMPLIANCE

Using K = 1.5 and a modification factor of 0.93 for the existing concrete building, the total seismic base shear V is 486 kips in the longitudinal direction, 415 kips in the lateral direction. Stiffness analysis should be carried out with the inverted triangular distribution of seismic forces to determine individual member stresses, but because of time constraints approximations are made here by assuming uniform distributions.

In the lateral direction the total seismic force is less than the total wind load, so it is reasonable to consider the steel bracing also adequate to resist seismic forces. In the longitudinal direction the seismic forces are considerably higher but may still be within allowable limits. Using an ultimate strength and load factor analysis, the lowest level columns, which are probably the most highly stressed, appear to be loaded to just below their capacity. The slab/beam elements cannot be adequately assessed as insufficient information is available about their reinforcement. However, the original building appears to have been designed for 100 psf live load, which is now reduced to 40 psf plus partitions. A combination of seismic forces and the new live load gives negative moments at the supports about 12% higher than the original gravity loads only. At this level, it is likely that more information about the slab reinforcement would allow an engineer to justify the strength of the existing section.

If the reinforcement in the slab were not adequate, then there are two likely design solutions. The first would be to add some steel bracing similar to that in the lateral direction. The major impact of this would not be financial but architectural; the braces would seriously interfere with the floor layouts and would probably not be feasible. The second possible solution would be to add one less floor to the building. This would reduce W in the determination of V by only 5%, but by changing the increase in area to 50% and allowing a modification factor of 0.80 instead of 0.93, V is reduced by a total of about 18%, which would probably bring stresses within allowable limits.

4.7 COST ESTIMATES

The total construction cost for the building was around $7.5 million. Bracing added within the existing structure would account for about $150,000 of this - about 2% of the total cost.

It is likely that meeting the new requirements would not change the cost. A more detailed analysis could probably be made to justify the scheme as it stands. If not, the worst case would be to add a similar amount of bracing in the other direction, with a similar cost.

The structural engineering design fee of $60,000 already included some investigation of the structure and lateral load analysis. The scope of both would need to be increased somewhat to meet the new provisions and this would probably add about $4,000 to the fee.

4.8 COMMENTS AND DISCUSSION

The code requirements at the time were probably misinterpreted in this case, though they were vague and probably not fully understood by many. Even then, the only realistic approach would have been similar to that advocated by the new Section 3208, documentation of which was probably available at the time without being widely known. Obviously its inclusion in the Code clarifies the process here.

It is not clear whether the new Section 3208 would require more extensive retrofitting work than was done, but it is clear that far more detailed investigation and analysis would be required in order to establish this. If any further action were necessary it is likely that it would be in the form of a slight reduction in scale rather than the additional cost of adding architecturally unacceptable bracing. The effect of Section 3208 on the design process would be greater than that on construction.

Part 5

LOWELL DYE HOUSE

5.1 BUILDING DESCRIPTION

The Dye House on Cabot Street in Lowell, part of the Wannalancit Mill Complex, is typical of late 19th century New England mill construction using load bearing brick exterior walls and wood floors.

The building consists of a three story portion 160 ft. by 35 ft. with a pitched roof, and a flat roofed two story portion 160 ft. by 50 ft. The two story part of the building was added to the original three story boarding house early in the life of the building, with a row of columns replacing one exterior wall. A typical floor plan and building section are shown in Appendix D.

Typical floor construction is 3" tongue-and-groove wood planking on 10" x 15" wood beams spaced at 8' and supported by 8" (square and round) wood columns spaced at 17' and 26'. Some of these columns go through the third floor to support the pitched roof framing on braced T-frames. Exterior brick walls are 16" thick. Some more recent single story ancillary buildings had been built up to this structure, but not tied into it.

5.2 PROPOSED ALTERATIONS

In 1988, when design work commenced, the building was unoccupied but its previous use was as a dye manufacturing factory. It was proposed to modify the building to use the first two floors for retail space with a central atrium cut in the second floor, and the third floor for offices. In addition the ancillary buildings would be demolished to allow construction of a new cinema building tight alongside the main building.

This scheme reached about 90% completion before being abandoned. In 1993 a new developer took over the project and proposed new uses: a microbrewery on the first floor, and a recreation center with video games and miniature golf taking up the second floor and sharing the third floor with a restaurant.

5.3 CONTEMPORARY CODE REQUIREMENTS

Under the requirements of what was then Article 22 of the Massachusetts State Building Code, the addition should not impose stresses on the existing building exceeding those allowed for new buildings. Further requirements are based on use group hazard indices.

The old use of the building for dye manufacturing is attributed a hazard index of 3 or 8 depending on the solvents used, based on consideration of fire hazard. A hazard index of 3 seems appropriate for our purposes. The 1988 design with retail and office space would not have increased the hazard index, so no further consideration of seismic resistance was required.

The proposed uses in 1993 cause a much more complex situation, and the process becomes muddied. Manufacture of alcoholic beverages should really require a hazard index of 8 as a high fire hazard, but this is probably not intended to apply to seismic requirements. A hazard index of 3 seems appropriate. The recreation center has a hazard index of 4 and the restaurant 5. This gives us no increase in hazard index on the first floor, an increase of one on the second floor, and an increase of two for part of the third floor. Each of these cases has a different requirement, and the only guidance given by the code for such a situation is related to fire separations and makes no sense when applied to requirements for earthquake resistance. The rationale used in this situation was to take a representative average and consider the building's hazard index to be increased by one, which requires only that the lateral load resistance of the building should not be reduced.

5.4 METHODS OF COMPLIANCE

Floor beams were reinforced with steel channels to support the increased live load.

From a seismic viewpoint, the cinema addition is self-supporting, with several reinforced concrete masonry walls. The floors are precast concrete plank or concrete on metal deck and steel beams. The new structure does bear on one wall of the existing building where steel beams were pocketed into the brick walls. Steel straps were provided to tie the existing wood beams through the wall to the new steel beams. This arrangement was believed to improve the overall lateral load resistance of the building, certainly to neither reduce it nor increase stresses in the existing building.

Two other modifications were made that require consideration. An 18' slot was cut through the center of the second floor slab, but as there is at least 30 ft. of floor between the opening and all of the exterior walls, this was considered not to reduce the ability of the floor to act as a diaphragm, at least to the extent that its connections to the walls would allow. The attic floor in the three story portion was in poor condition and was removed. A steel bracing and tying system was installed to replace the floor's diaphragm action.

5.5 REQUIREMENTS OF NEW SECTION 3208

The cinema building is a structurally attached addition, and increases the weight of the building by about 50%, and the area by about 32%. Two interesting points are raised in considering the requirements of Section 3208.4.3.2. With the increases noted above the building falls under the second category and requires a percentage of the base earthquake force to be obtained from figure 3208.1. I assume this should consider the higher figure, in this case the 50% increase in weight which requires 80% of the base earthquake force, but no requirement to use the larger figure (rather than the lower of the average) is stated. The second point is whether or not an unused attic space is considered part of the "area". In some cases, considering it as such could be favorable since it is then removed, and the resultant net increase in area is here reduced to 13%, but is not pertinent as the weight increase governs.

The building also has a change of use and must be considered under Section 3208.5.4. Table 3208.1 determines the seismic hazard category. The occupancy of the building is increased by more than 25% and to more than 100, which gives a category of 2 or 3 depending on change in use group. The first floor (on grade) stays at hazard index 3, giving a seismic hazard category of 2. The second and third floors have a hazard index increased from 3 to 4 or 5; in either case category 3 applies. Section 3208.5.4.2 deals with partial changes of use and invokes the higher hazard category when it applies to more than 33% of the area, which it does in this case.

Seismic hazard category 3 requires full compliance with Section 1113.0, but existing systems not meeting the ductility detailing requirements of Section 1113.5 may be considered with a magnified ductility factor K taken from Table 3208.2, equal to 4.0 for an unreinforced masonry bearing wall system. These requirements are greater than those arrived at through Section 3208.4.3.2 covering additions, and therefore govern. Section 3208.6.4 applies, but in this case its requirements (that the wall be of sufficient strength to resist the forces required by Section 1113.4.5, and be adequately tied to its lateral supporting elements) seem to be redundant as the requirement for full compliance with Section 1113.0 includes 1113.4.5 which covers both of these requirements.

Another very pertinent requirement of Section 3208 is the structural evaluation of the building (3208.2) including field surveys, testing and laboratory analysis. Key items in this building would be brick and mortar strength, and floor to wall connection details.

5.6 POSSIBLE METHODS OF COMPLIANCE

The existing building and the new addition will be considered separately for seismic purposes; the new addition meets full requirements for new construction and will be considered to not contribute to the existing building's lateral load resistance.

The existing building, with a K value of 4.0, has a base earthquake force of 348 kips in its long direction, and 313 kips in its short direction, producing masonry shear stresses of 15 psi and 16 psi respectively. These are within allowable limits, although it is unclear what is the actual allowable stress, as discussed in Part 7. There is no positive connection of the floor diaphragm to the walls, and indeed the floor may not be adequate to act as a diaphragm. The force required to be transferred from the wall to the floor (in tension) is 100 pounds per foot using Table 1113.2 with $C_p = 0.2$, but 400 pounds per foot using figures from Section 1113.4.2.

I found note 3 to Table 1113.2 confusing but apparently relevant. This is also discussed in Part 7.

To achieve this force transfer, steel straps would be fixed to the floor framing and attached to threaded rods passing through the walls with plate washers on the outside. It would be convenient to use one such connection at each joist location, 8' apart giving a force of 3,200 pounds. Appendix F shows typical details appropriate for this application.

The floors could be made effective as diaphragms by nailing plywood, or casting a concrete topping, over the plank floor. In addition, tension chords around the diaphragm may be required, which could be achieved by steel angles screwed or bolted to the floor framing. Also, some reinforcement would be required around the second floor opening.

5.7 COST ESTIMATES

The construction cost was about $2 million, half of which was in the existing part of the building.

Adding new floor diaphragms and tying them into the masonry walls would add about $60,000 to this cost, which represents about 3% of the total project cost or 6% of the work on the existing building alone.

A more detailed investigation including some testing, and a seismic analysis could increase the structural design fees from $20,000 to $25,000.

5.8 COMMENTS AND DISCUSSION

The new code requires more to be done for seismic resistance than its predecessor in this case, but its requirements do not seem unreasonable and their economic effects are unlikely to be prohibitive for the project.

Several points of interest arise from consideration of this project. The effects of using hazard index 8 for high hazard uses, the allowable shear strength of brick masonry and the use of Table 1113.2 and its notes to determine transfer forces at wall to floor connections are all discussed in more detail in Part 7.

In the case of this building, a designer could assign a hazard index of 8 (for chemical dye manufacture) to the building before renovation, and thus reduce the seismic hazard category to 2.

This would require wall to floor connections to be designed for only 100 pounds per foot.

The new Section 3208 resolves the confusion surrounding partial change of use but one question arises from Section 3208.5.4.2. Should floors supported directly on the ground be included in the area calculation? The answer appears to be "Yes", both from a reading of that Section (which does not say otherwise) and logically, as a space is affected by the structure above it as well as below. This may need to be clarified however.

It is interesting to consider alternative ways the problem of compliance with the new Section 3208 may be considered. If the project were designed with Section 3208 enforced, the addition would probably be kept structurally separate. This would allow the design lateral seismic force to be reduced by 25% under Section 3208.5.4.6, thus reducing the required tie force at the floor to wall connection to 300 pounds per linear foot, but probably not changing the detail.

This could be further reduced to 100 pounds per linear foot by restricting the building to use groups of hazard index 3 or lower, which was the situation for the original project in 1988. Interestingly, this would probably not result in a substantial cost savings as the connections would still need to be made every 8 feet.

Part 6

NORTH READING LIBRARY

6.1 BUILDING DESCRIPTION

Flint Memorial Hall was constructed in 1875 in the center of North Reading. It is a 3-story wood framed building with a mansard roof and stone foundation walls, 65' by 50' in plan. The lower two floors were framed with 2 x 12 joists spanning between the exterior and two wood interior bearing walls running the length of the building. The second floor was a two story high space. The attic floor and roof were carried by six heavy timber and steel rod trusses with wood purlins. A building section and floor plan are included in Appendix D.

Prior to 1990, the building was used as the Town Hall and offices.

6.2 PROPOSED ALTERATIONS

The design developed in 1990 was to convert the building for use as the town library. In addition to gutting the interior of the building, removing partitions and ceilings, existing floors needed to be upgraded structurally to carry library loading. Steel beams and columns replaced the interior bearing walls, and floor joists were doubled up. A new mezzanine level floor was added in the two story space to replace the small existing balcony. The existing attic floor was retained, but not used structurally; a new floor was framed over it. A two story extension, 45' by 30' was added at the rear (east) of the building, together with an elevator and stair on the north side with solid grouted reinforced masonry walls.

6.3 CONTEMPORARY CODE REQUIREMENTS

As the hazard index was increased from 2 to 4 the building had to meet the requirements for new construction. As wood does not have such severe detailing requirements as concrete and masonry, this did not present a major problem.

6.4 METHODS OF COMPLIANCE

Masonry walls are often the most efficient construction for elevator and stair shafts for non-structural reasons. By grouting and reinforcing these walls and tying them into the existing building they could carry the lateral load in a north-south direction. In the east-west direction, blocked plywood sheathing was added in place of the original lath and plaster on the interior face of the exterior wood stud walls. New anchors were provided to better tie the wood walls to the stone foundation walls. Shear stresses in these stone walls were considered low enough to be acceptable.

It should be noted that wind load governed due to the overall lightweight nature of the building. The new additions were structurally separated as far as gravity loads were concerned.

6.5 REQUIREMENTS OF NEW SECTION 3208

As the new additions increase the weight by over 100%, the requirements for new construction would apply. Even if the additions were kept totally separate structurally, the cost of alterations in the existing portion of the building would exceed 50% of its value, and the hazard index being increased to 4 would give a seismic hazard category of 3. This also requires full compliance with the Code for new construction, except for the detailing requirements, the need for positive connections in wood construction.

6.6 POSSIBLE METHODS OF COMPLIANCE

The requirements of the new and old codes are essentially the same in this case, and the system used would still be appropriate.

6.7 COST ESTIMATES

The total construction cost was about $3 million, one-third of which could assigned to the extension. The value of work carried out purely for lateral load resisting purposes in the existing building was probably less than $20,000. This small proportion - about 1% - of the total costs can be attributed to the fact that the large amount of work allowed a lateral load resisting system to be easily developed by making use of elements already included for other reasons.

There would also be no additional design costs here. The fee of around $25,000 included lateral load analysis, as it was clear that it would be required by the existing Code given the nature of the project.

Both construction and design costs for a lateral load resisting system are low on this project mainly as a result of its construction type. A wood framed building is both seismically efficient (relatively high resistance but low weight and hence force) and comparatively simple to evaluate. In addition the extent of other work tends to overshadow the lateral load considerations in much the same way it would for wholly new construction.

6.8 COMMENTS AND DISCUSSION

For this building, the requirements of the existing code are both severe and explicit, but due to the extent of the new construction and the type of building, are not difficult to meet. This would still be the case with the new provisions, though a little more study is required to establish that this is the case.

Part 7

ITEMS FOR DISCUSSION

7.1 HAZARD INDEX

The use of hazard indices listed in the Code is not so important in the new Section 3208 as in the current Section 32, where they are the major parameter governing work on existing buildings. Hazard index is only relevant under Section 3208 if it changes from 3 or less to 4 or more. This is intended to cover situations where buildings are to be used for assembly or high occupancy purposes that previously were not. The hazard indices, originally based on fire hazard, seem to fit this need fairly well.

However, the allocation of hazard index 8 to high (fire) hazard uses for seismic considerations tends to cause the system to break down. Significantly, it can work both ways. A building that has its use changed to one with a high fire hazard would also have severe seismic requirements. This may be unreasonable but at least it is safe. The opposite situation is of more concern. An industrial building with high fire hazard being converted to a theater, nightclub or other assembly building would actually have a reduced hazard index, and therefore fall into seismic hazard category 1 or 2. This does not seem to meet the intent of Section 3208.5.4.

Some consideration should be given to this situation. The easiest answer seems to be to eliminate hazard index 8 in seismic considerations and to apply the appropriate hazard index for the same use group neglecting its high fire hazard. In most cases this would be 3, as all high hazard classifications seem to be in industrial situations. A more conservative approach would be to apply this only to existing uses, but this may impose onerous restrictions. A more rigorous approach would be to define seismic hazard indices, but this seems to be an unnecessary extra task.

To be comfortable about removing hazard index 8, one would need to be sure that a building was not also a high seismic hazard. The only situation that comes to mind is the manufacture or storage of explosives, but such buildings generally have their own blast resistance requirements that would far exceed seismic requirements.

7.2 BRICK MASONRY SHEAR STRESS

The allowable shear stress of unreinforced brick masonry with lime mortar under seismic loading is a figure that is rarely used by engineers now, and the question arises of what it should be. The Brick Institute of America's "Building Code Requirements for Engineered Brick Masonry" (BIA-69) seems to be applicable but is not listed in the Massachusetts Code's referenced standards in Appendix A. It is referenced by the BOCA Code. A more recent publication "Building Code Requirements for Masonry Structures" (ACI-531) gives allowable stresses that seem to be based on concrete masonry and are somewhat higher than those in BIA-69. It is questionable whether these are appropriate for the circumstances encountered here, of older brick masonry construction. ACI-531 is similarly not referenced by the Massachusetts Code. Some clarification seems to be necessary as to which standards are applicable.

7.3 FLOOR TO WALL FORCE TRANSFER

In considering the force required to tie masonry walls into their restraining floor diaphragms, Table 1113.2 becomes important. this gives a value for C_p of 0.20 to be used in the formula $F_p=1/3C_pW_p$ but also references note 3. Note 3 cites the need for higher forces to be used if "required by the basic seismic formula V=1/3(KCSW) and the coefficient F_x/W_x [the dividing line is omitted] at the height h_x...." For usual values of K, no greater than 1.33, this clause will have little effect, but with K equal to 4.0 for unreinforced masonry it becomes pertinent but unclear. From a study of various other sources I understand it to require $F_p=(F_x/W_x)\ W_p$ but cannot see how this could be inferred from reading note 3. Note 4, referring to diaphragms, appears to suffer from a similar problem.

Beyond their lack of clarity, the effect of these notes merits further examination. With high K values, the numbers obtained for F_p can exceed 500 pounds per foot. Some consideration should be given as to whether or not this is the intent of the new provisions. If these forces are indeed higher than intended, perhaps a force modification factor could be applied, or a blanket figure of 200 pounds per linear foot.

7.4 APPLICABILITY (SECTION 3200.3)

Item 10, structural requirements, seems to be intended to override the other items in this paragraph, but this is not clear. This is particularly so in item 3 which refers to a change in use group with increase of two or more in hazard index. In item 9, the Section for Historic Buildings should be 635.0 not 436.0

7.5 INCREASE IN WEIGHT AND AREA (SECTION 3208.4.3.2)

When additions are not structurally separated, the percentage of lateral earthquake load to be applied is derived from figure 3208.1 based on the increase in weight or area. Presumably the larger figure should apply, but this is not stated.

Item 4 here defines "area" as the total of all gross floor and roof areas supported by the building structure. It is not clear whether unused attic floors should be included. This situation occurred in two of the five study buildings, one where apartments were added to an attic floor, another where an unused attic was removed. Excluding unused attic floors from the area calculations is more severe, but probably reasonable. Occupation of a previously unused attic space would thus constitute an increase in area, while removing an unused attic space could not be considered a reduction in area to offset other increases.

7.6 PARTIAL CHANGE OF USE (SECTION 3208.5.4)

For partial changes of use, the area is presumably calculated as gross floor space, not supported floor and roof as defined in 3208.4.3.2 item 4.

7.7 REDUCTION OF EARTHQUAKE HAZARDS (SECTION 3208.6.3)

This section appears to be included as a minimum requirement to improve a building's seismic resistance when the size and nature of alterations are considered not to warrant a full seismic analysis with modified K values.

Although the situation was not encountered in the buildings studied, there seems to be an anomaly regarding the item about precast concrete structural elements. There are two possible situations where the new provisions would require full compliance with the code for new construction and allow a modified K value (1.5 or 1.33) in lieu of meeting the ductility requirements of Section 1113.5, but not require the reduction of special earthquake hazards. The two situations are when the building's lateral load capacity is reduced and when a seismic hazard category of 3 applies because of change of use and the value of alterations.

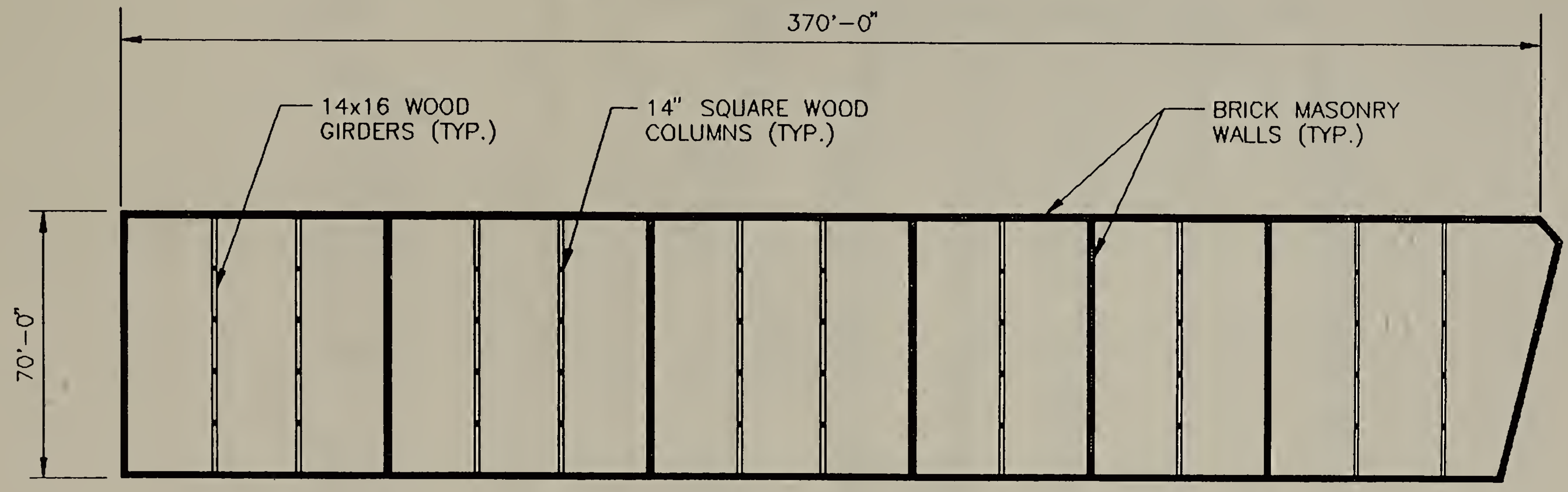
In these situations, the new provisions do not require connections to conform to Section 1113.5.5 (and by further reference, 1113.5.7), but in the supposedly less severe case of a seismic hazard category of 2, they do.

If this is indeed contrary to the intent of the new provisions, it could be rectified by requiring the correction of special earthquake hazards whenever calculation of earthquake design forces is required, that is, in Section 3208.6.1.1. The requirements for parapets and masonry walls seem to be covered by implication, but including them explicitly as well would do no harm and probably be beneficial.

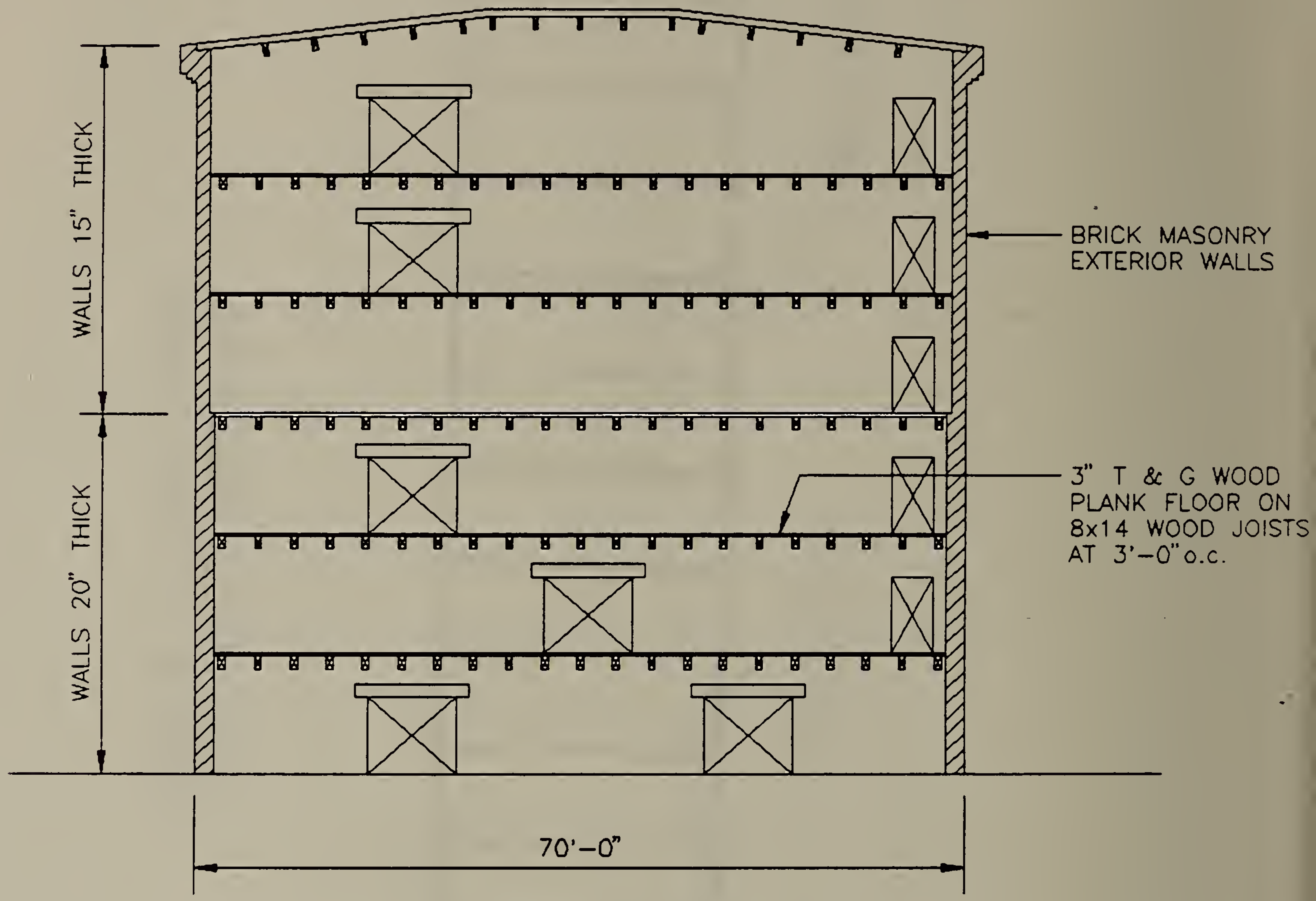
APPENDIX A

CHILDRENS MUSEUM

CALCULATIONS AND ILLUSTRATIONS


370'–0"
14x16 WOOD GIRDERS (TYP.)
14" SQUARE WOOD COLUMNS (TYP.)
BRICK MASONRY WALLS (TYP.)
70'–0"

CHILDRENS MUSEUM
TYPICAL FLOOR PLAN


WALLS 15" THICK
WALLS 20" THICK
BRICK MASONRY EXTERIOR WALLS
3" T & G WOOD PLANK FLOOR ON 8x14 WOOD JOISTS AT 3'-0"o.c.
70'-0"

CHILDRENS MUSEUM
BUILDING SECTION AT TYPICAL CROSS WALL

BOSTON BUILDING CONSULTANTS

Subject: CHILDRENS MUSEUM Job No: Sheet: 1

By: ACA Date: 1/94 Checked:

Building Weight

Floors	3" plank	10
	beams, girders	15
	services, etc	5
		30 psf

Roof	2" plank	7
	structure	8
	roofing, etc	5
	½ snow	15
		35 psf

Walls	1st-4th	20" thick		200 psf
	say 85% solid	33' high		5.61 klf
	4th-roof	15" thick		150 psf
	85% solid	34' high		4.34 klf

Typical floor area 370 × 70 = 25 900 sf

plan - walls	exterior	(370 + 70) × 2 =	880 lf
	interior	70 × 5	350 lf
			1230 lf

Total weight -	5 floors @ 25900 × 30 psf	3885 k
	roof 25900 × 35 psf	907 k
	20" walls 1230 lf × 5.61 klf	6900 k
	15" walls 1230 lf × 4.34 klf	5338 k
	W =	17030 k

Subject: CHILDRENS MUSEUM
Job No:
Sheet: 2
By: ACA
Date: 1/94
Checked:

In long direction — short direction

D = 370'	D = 70'
h_n = 67'	h_n = 67'
T = 0.174	T = 0.400
C = 0.090	C = 0.068
S_3 = 1.5	S_3 = 1.5

$V = \frac{1}{3} KCSW$

$= \frac{1}{3} \times 4 \times 0.09 \times 1.5 \times 17030 = 3065$ k long direction

$0.068 = 2316$ k short direction

In long direction:

resisting walls - about 70% of 2×370' @ 20" thick

f_v = 25 psi

In short direction

resisting walls - about 70% of 7×70' @ 20" thick

f_v = 28 psi

allowable stresses -

using $\left[0.5\sqrt{f'_m} + 0.2\,(DL)\right] \times 1.33$ (max = 28 × 1.33

f'_m = 1500 psi

in long direction - dead load from wall only

9.95 klf - 41 psi

F_v = 37 psi

in short direction - dead load higher - includes floors

F_v = 37 psi

BOSTON
BUILDING
CONSULTANTS

Subject: CHILDRENS MUSEUM
Job No:
Sheet: 3
By: Alxi
Date: 1/94
Checked:

Distribution for each floor

level	H	W floor	W wall	W Total	$W_x h_x$	F_x long	F_x short
2nd	11	777	2300	3077	33847	184	139
3rd	22	777	2300	3077	67694	368	278
4th	33	777	2013	2790	92070	490	371
5th	44	777	1725	2502	110 088	582	440
6th	55	777	1725	2502	137 610	736	556
R	67	907	1019	1926	129 042	705	533

Using V : 3065k in long direction
2316 k in short direction

At 5th floor level

In long direction V = 2023 k
f_v : 22 psi

allowable stress : 29 psi

In short direction V : 1529 k
f_v : 25 psi

allowable stress : 37 psi

Force to be transferred at floor/wall connection

Using section 1113.4.5

(i) from Table 1113.2 - direct $C_p = 0.2$ for walls
$C_p = 0.1$ for floors (W_p = loads from that story)

(ii) from notes to Table 1113.2
"using $V = \frac{1}{3} KCSW$ and F_x/W_x" - i.e. using F_x instead of W

(i) - Consider a typical exterior wall at a low level, for a 1' length

20" wall - 11' trib height - $W_p = 2.2$ k

using $F = \frac{1}{3} C_p W_p = 150$ #/lin ft

higher levels - $W_p = 1.65$ k
$F = 110$ #/lin ft
- these are tie forces.

Considering floors: this gives a shear force.
The critical shear force will be in the long direction only as in the short direction the shear force is transferred by direct bearing of the sides of the joists in the wall

consider a 70'x70' panel of floor including a cross wall

for each 1' length of wall: tributary load is
35' floor @ 30 psf - 1.05
1'x11' wall @ 200 psf 2.2
3.25 k.

$F = \frac{1}{3} C_p W_p$ with $C_p = 0.1$
$F = 108$ #/lin. ft.

(ii) Using $V : \frac{1}{3} kCSW$ and distribution of forces using F_x/W_x

here, the upper levels will be critical

consider 6th floor - $W_x : 2502$ k
$F_x : 736$ k in long direction

$F_x/W_x : 0.29$

Exterior walls - tension force pulling wall out.

use wall weight : 1.65 klf

$F : 0.29 \times 1.65 : 480$ # / lin. ft.

Shear force - from diaphragm - $W : 2.7$ klf

$F : 0.29 \times 2.7 : 780$ # / lin. ft.

Consider a lower level - 3rd floor -

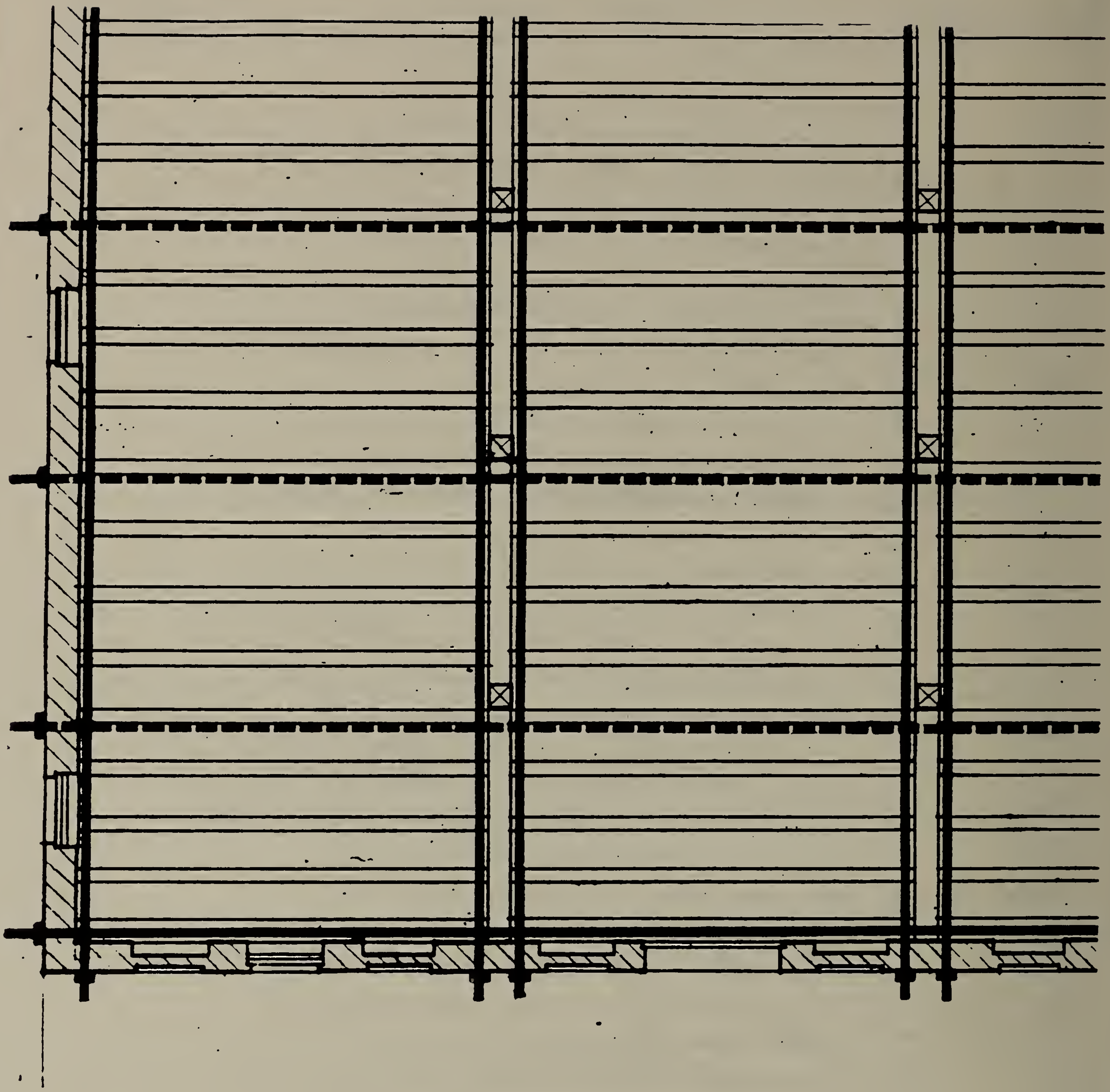
$F_x/W_x : 0.12$

wall $W : 2.2$ klf
$F : 260$ #/lin.ft

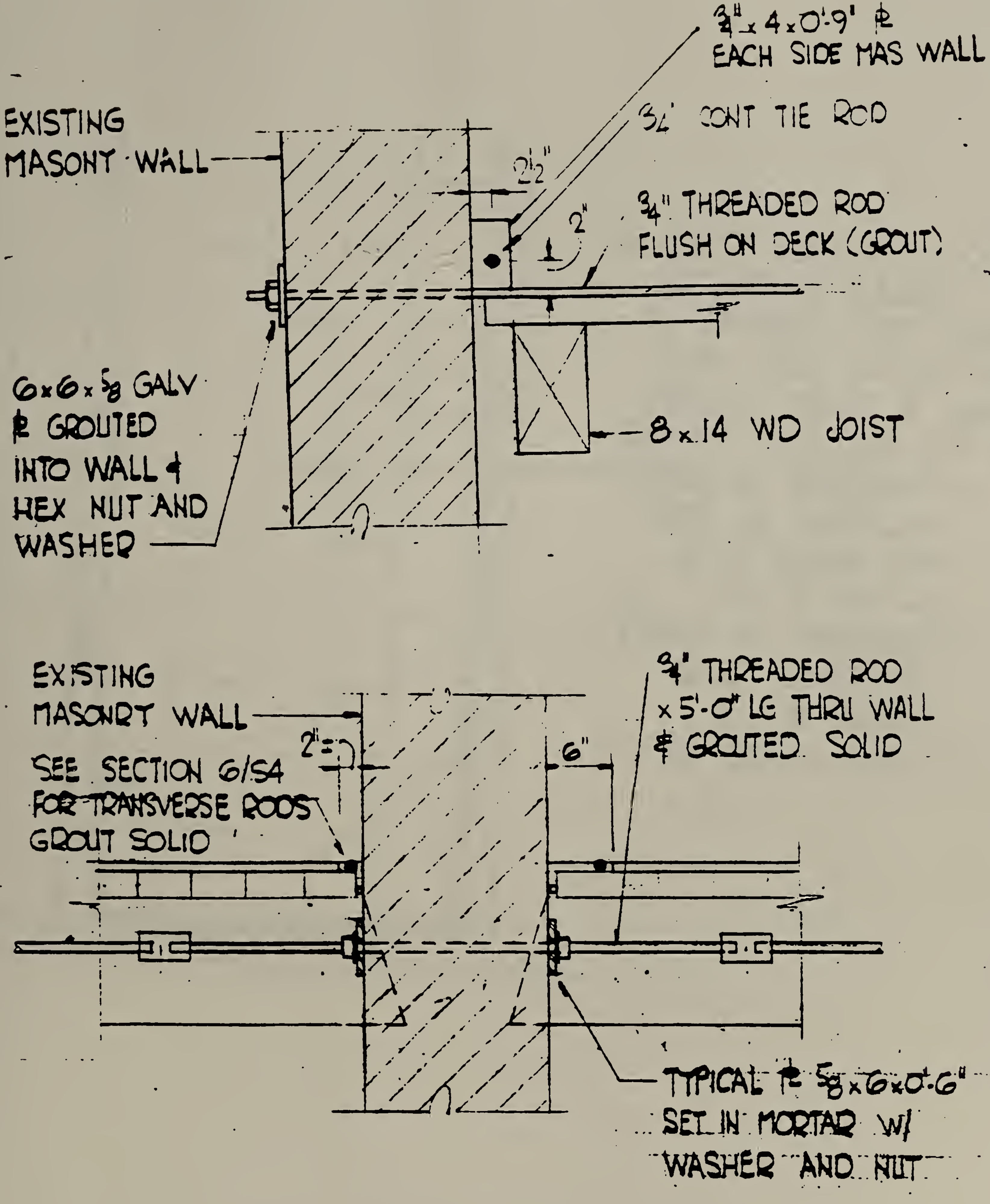
floor diaphragm $W : 3.25$ k
$F : 390$ # /lin. ft

Summary - typical key numbers

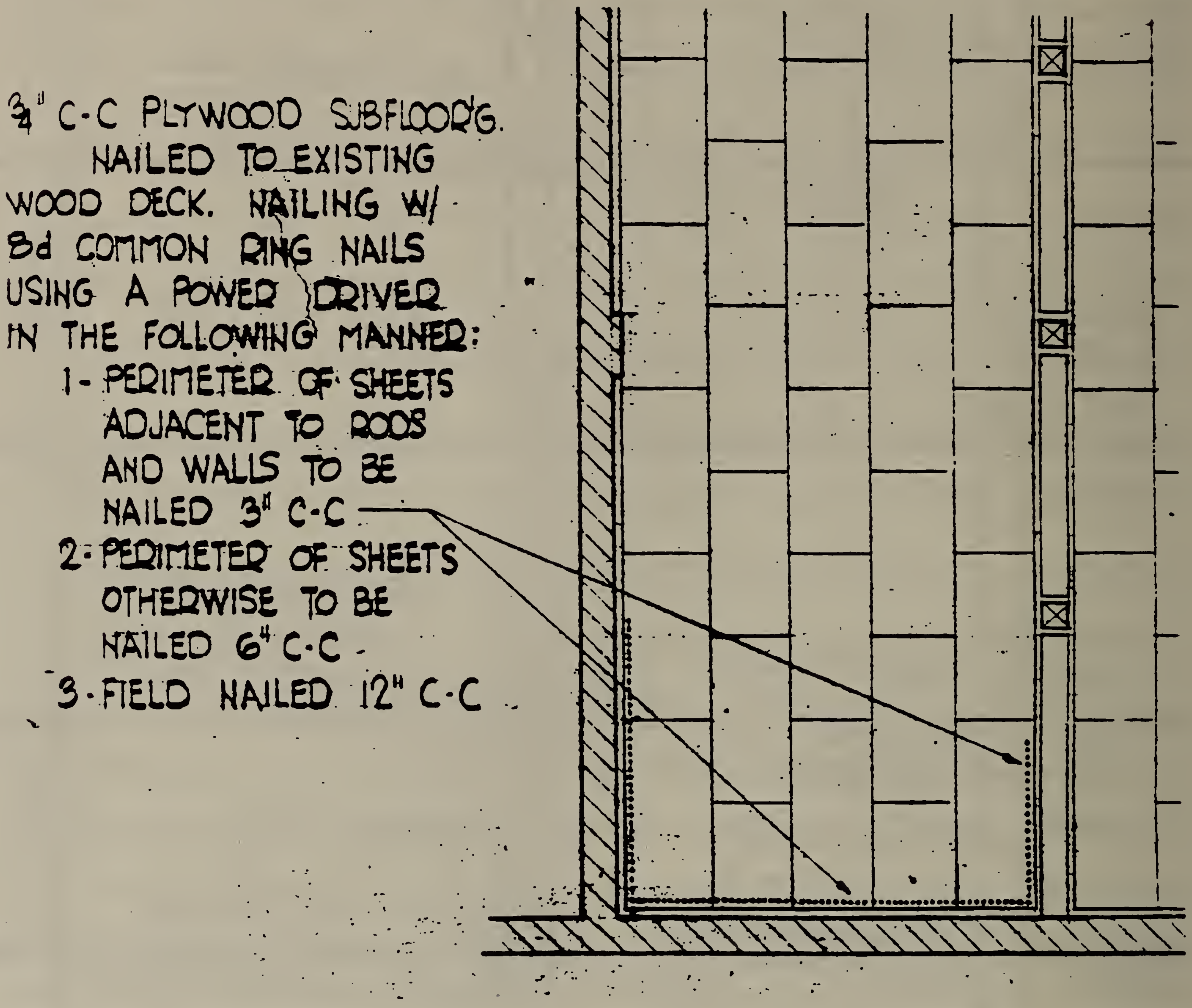
	Tension - wall		Shear - diaphragm	
	Low	High	Low	High
$F : \frac{1}{3} C_p W_p$	150	110	108	< 100
F_x/W_x W_p	260	480	390	780



PART PLAN - TYPICAL FLOOR
TIE ROD LAYOUT



TYPICAL TIE ROD DETAILS

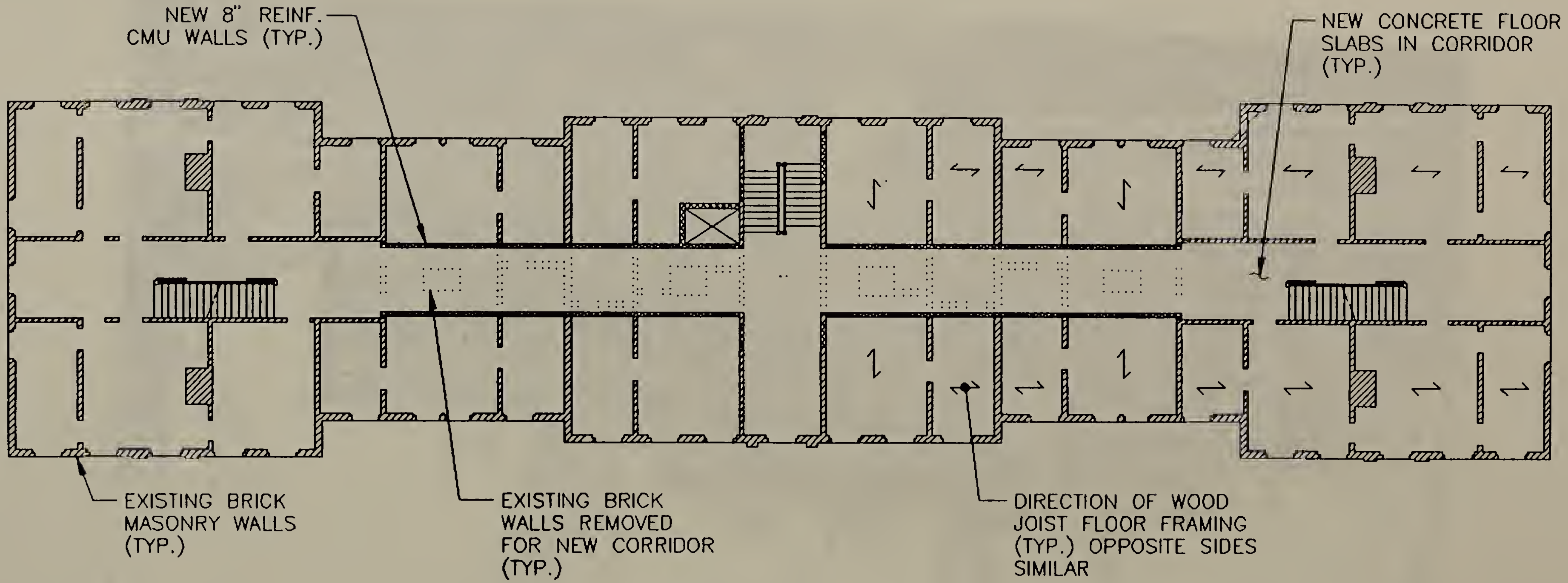

3/4" C-C PLYWOOD SUBFLOOR'G. NAILED TO EXISTING WOOD DECK. NAILING W/ 8d COMMON RING NAILS USING A POWER DRIVER IN THE FOLLOWING MANNER:
1- PERIMETER OF SHEETS ADJACENT TO RODS AND WALLS TO BE NAILED 3" C-C
2- PERIMETER OF SHEETS OTHERWISE TO BE NAILED 6" C-C
3- FIELD NAILED 12" C-C

PART PLAN - TYPICAL FLOOR
NEW PLYWOOD DIAPHRAGM

APPENDIX B

THAYER HALL

CALCULATIONS AND ILLUSTRATIONS


NEW 8" REINF. CMU WALLS (TYP.)
NEW CONCRETE FLOOR SLABS IN CORRIDOR (TYP.)
EXISTING BRICK MASONRY WALLS (TYP.)
EXISTING BRICK WALLS REMOVED FOR NEW CORRIDOR (TYP.)
DIRECTION OF WOOD JOIST FLOOR FRAMING (TYP.) OPPOSITE SIDES SIMILAR

THAYER HALL
TYPICAL FLOOR PLAN

Subject: THAYER HALL
Job No: 92062
Sheet:
By: AGA
Date: 2.10.93
Checked:

Lateral load analysis

Using the proposed new section 3208 of Mass code - to be introduced this summer.

Building is being altered, but not really extended. Increase in weight of the building will be small - more walls being removed than added, and concrete slab added to replace wood. If less than 10% - than compliance with alteration requirements only.

Lateral load analysis needs to be analysed because some of potential resisting elements are being removed. (3208.3.5 - Lateral load resisting system should meet sections 1112 & 1113 but with increased K factors from Tables 3208.2, 3208.3 as building does not meet detailing requirements of 1113.5 - masonry not reinforced)

Wind loads - 3208.5.3. - for structure as a whole - not less than Exposure A

Seismic hazard category - occupancy not increased by more than 25%
total cost of alterations less than 50% assessed value
no change in use group

Table 3208.1 - S.H.C. = 1

∴ Need only comply with 3208.3.5 (- noted above)
ie. no need to bother with 3208.6.3. which requires bracing or removing of parapets (not an issue here) and connecting masonry walls to floor diaphragms (would be a big issue!)

3208.5.4.6. Building not extended in area or height - can use 75% of base earthquake force (assumes that converting attics to apartments does not constitute an extension of area)

From table 3208.2 - Unreinforced masonry bearing wall system - K = 4.0

No modification factors are required from Table 3208.3; although it is applicable we don't have any of those conditions listed.

Subject: THAYER HALL
Job No: 92062
Sheet: 2
By: ACA
Date: 2.10.93
Checked:

3208.6.4. Existing unreinforced walls in sound condition may continue in service providing they are adequately tied to structural elements providing their lateral support and $l/t \ngtr 20$. (see note below)

3208.7 For seismic hazard category 1 - no evaluation of liquefaction potential of soil is required.

Summary :

1. Ensure that weight is not increased by more than 10%
2. Find base earthquake force with K = 4 & mod. factor of 0.75
3. Use wind load exposure A

Note : Floor joists are built in to masonry walls but in all likelihood not positively tied in. The code requirements in this circumstance are to provide restraint for the walls at the floor levels. Interior walls have floor both sides, and therefore are adequately restrained. Exterior walls do not form a significant part of the lateral load resisting system anyway, but are tied into the perpendicular interior walls at intervals of 15 feet or less, so these also are not likely to pull away.

BOSTON
BUILDING
CONSULTANTS

Subject: THAYER HALL

Job No: 92062

Sheet: 3

By: ALA

Date: 2.11.93

Checked:

Building weight W - in kips		Existing to remain	To be removed	To be added	
Basement walls (11' high)	8" brick 80psf 55/20 lf	48	18	-	
	12" brick 120psf 450/150 lf	594	198	-	
	20" stone - exterior 200psf - 540 lf	1188	-	-	
	8" reinf CMU 60 psf - 180 lf	-	-	119	
	12" non-reinf. CMU 70 psf - 90 lf	-	-	70	
		1830	216	189	
Walls at typical floor central area (5 floors) 11' high	12" brick exterior - 120psf 165/14 lf	218	18	-	
	8" brick - 80psf 120/80 lf	106	70		
	8" reinf. CMU - 60psf 100 lf	-	-	66	
		324	88	66	
Walls at typical floor end areas (4 floors) 11' high	12" brick exterior - 120psf 340 lf	449	-	-	
	8" brick - 80psf 284/94 lf	250	83	-	
	8" reinf. CMU - 60psf 95 lf	-	-	63	
		699	83	63	
Typical floors central area (5 floors)	Ex. wood construction 30psf 1600/700 sf	48	21	-	
	New conc. construction 80 psf 600 sf	-	-	48	each
Typical floors end areas (4 floors)	Ex. wood construction 30psf. 4500/1200 sf	135	36	-	
	New conc. construction 80psf 1200 sf	-	-	96	each
Attic floors	Ex wood construction 20 psf 7000/1600 sf	36/104	9/27	-	
	New conc construction 80psf 1600 sf.	-	-	32/96	
Roof dead + ½ snow.	Flat roof : 15 + 15 psf : 2000 sf	60 (16/44)	-	-	
	Sloping roof (slate) - 30 + 10 psf - 8800 sf	352 (100/252)		-	
	4' high 12" perimeter walls - 540 lf	96/164	-	-	
	Existing chimneys - assume 8" 600/700 sf	48 (0/48)	56	-	
	New chimneys - 4" brick 700 sf	-	-	28 (14/14)	
	Chimneys - existing - 600/300	0/48	24/24		
	8" reif [illegible] - : lf	-	-	57/62	

BOSTON
BUILDING
CONSULTANTS

Subject: THAYER HALL
Job No: 92062
Sheet: 4
By: ACA
Date: 2.11.93
Checked:

Totals :

		To remain	Removed	Added
Basement		1830	216	189
Typical floor inc. walls above.	Central (5)	372	109	114
	Ends (4)	834	119	144
Attic floor inc. walls above	Central	132	33	132
	Ends	316	51	259
Roof inc. chimneys		460 (116/344)	56	28 (14/14)
		7934	1377	1754

W before alteration : 9311 k
W after alteration : 9688 k

Increase in W ' 4 % - within 10% . ok

(Addition is 19% of orig. weight but most is substitution)

For considerations of earthquake and wind loading, the long new reinf. CMU walls in the longitudinal direction are obviously a vast improvement on what is there.

In the lateral direction - analysis is required, as some walls are being partially removed.

Subject: THAYER HALL Job No: 92062

By: ACA Date: 2.11.92 Checked:

$V = \frac{1}{3}\ KCSW$ - total lateral force

$K = 4.0$ from table 3203.2

$S = 1.0$ Sand - not susceptible to liquefaction, over bedrock.

$C = 0.05/\sqrt[3]{T}$

$T = 0.05\,h_n/\sqrt{D}$

$h_n = 74'$

$D = 44'$

$T = 0.56$

$C = 0.06$

$W = 7669\ k$ - taken above basement as grade is virtually at 1st floor level. (W_{old} : 7265 W_{new} 7668 : 5.5% incr

$V = \frac{1}{3} \times 4 \times 1 \times 0.06 \times 7669$

$= 614$ kip

$F_t = 0.004\ V \left(\frac{h_n}{D_s}\right)^2$

$h_n/D_s = 1.7$ - F_t can be considered as zero

$Level_x$	h_x	w_x center	w_x ends	$h_x w_x$	F_x
High roof	74	130		9620	28 k
6th floor/low roof	60	264	358	37 320	108 k
5th floor	48	486	575	50 928	147 k
4th floor	36	486	978	52 704	153 k
3rd floor	25	486	978	36 600	106 k
2nd floor	14	486	978	20 496	59 k
1st floor	3	486	978	4 392	13 k

$\Sigma w_i h_i = 212\,060$

BOSTON
BUILDING
CONSULTANTS

Subject: THAYER HALL
Job No: 92062
Sheet: 6
By: ACA
Date: 2.11.93
Checked:

Check overturning. (this is oversimplified! - see below for alternative evaluation)

total overturning moment from one side of building at ground
= 24 631 k.ft

Restoring moment.
consider exterior wall on opposite side.
lever arm 44' - W = 865 k above 1st floor
M_e : 38 000 k.ft
- ok - rest of building will easily bring
M_e over $2 \times M_o$.

Shear.
Many brick walls will take the shear.

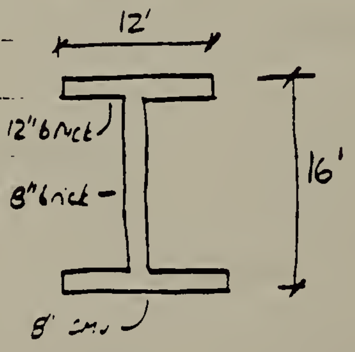
Total length of brick wall at a typical floor in lateral direction
330' - 8" wall
176' - 12" wall

At first floor level - total shear force = 614 kip

average shear stress in brick masonry : 11 psi - ok

Alternative evaluation of overturning :
considering only tied pieces of wall - in I configurations

36 pieces of wall thus



each with M_o : 684 k.ft
and 63' high

[illegible] M_e : 270 - factor of safety 2.0 - ok

BOSTON
BUILDING
CONSULTANTS

Subject: THAYER HALL

Job No: 92062

Sheet: 7

By: ACA

Date: 2.12.93

Checked:

For completeness - consider opposite direction

resistance is provided by reinforced CMU walls -
4 - 50' long = 110' long system

K = 1.0
D = 110'
T = 0.35
C = 0.07

V = 179 k - (29% of force in other direction)

∴ overturning moment = 7180 k.ft.

consider using 4 walls = 50' long. - each with 1795 k.ft

restoring moment each : 4725 k.ft - ok

Shear stress in walls = 9 psi - ok.

If K of 4 is used: existing brick masonry corridor walls in north and south wings could also be considered, as well as all of the exterior walls - so overturning could not be a problem. Even without them - shear stress in new CMU would be 36 psi. - ok.

Subject: THAYER HALL

Job No: 92062

Sheet: 3

By: ALA

Date: 2.12.93

Checked:

Wind loads

exposure A zone 3 - 14 psf.

building area : 210' × 70' (avg.) ?

F = 206 k.

- much less than earthquake load - ok

In other direction - area : 48' × 70' ×

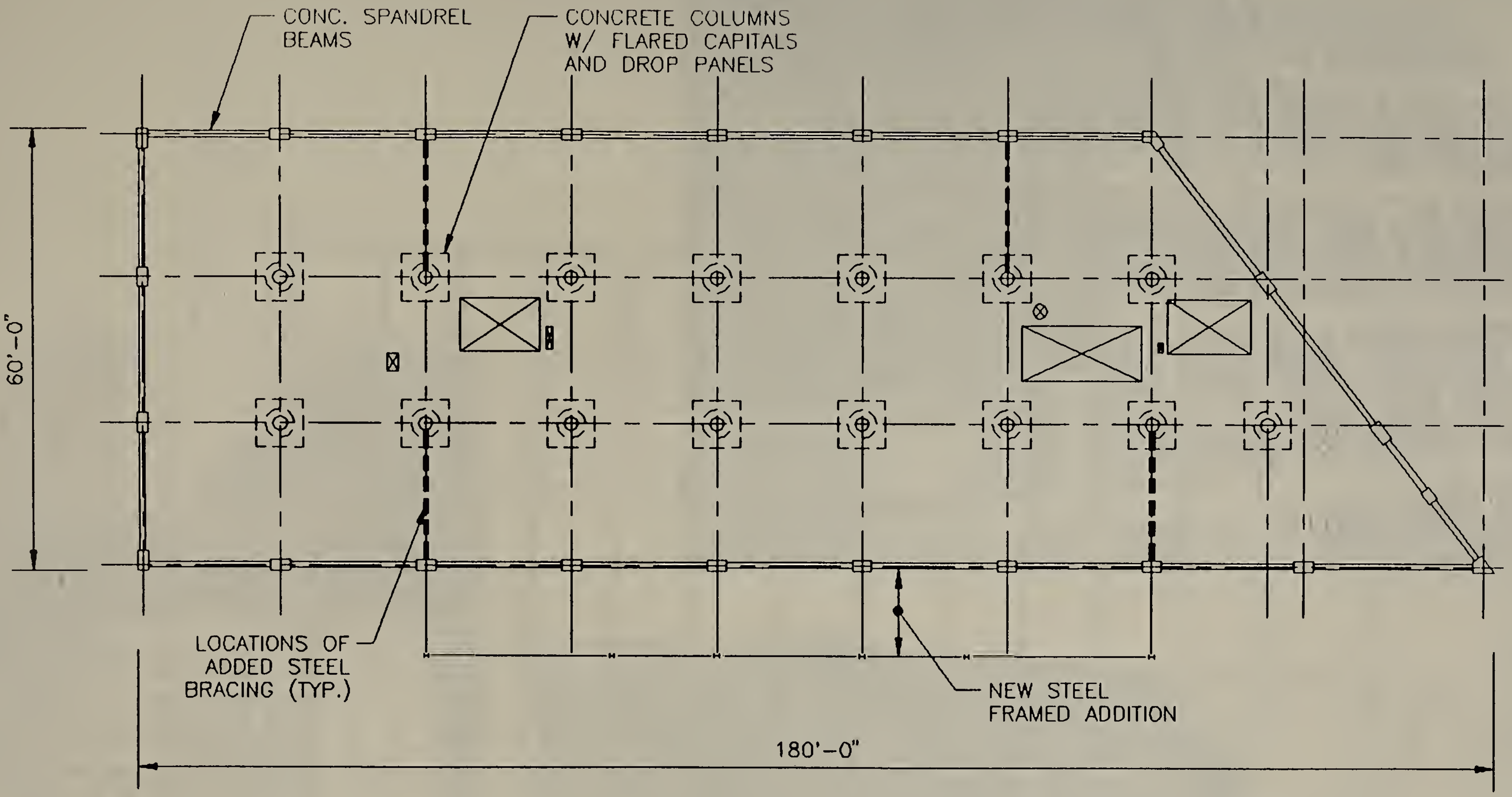
F = 47 k.

much less than earthquake load also.

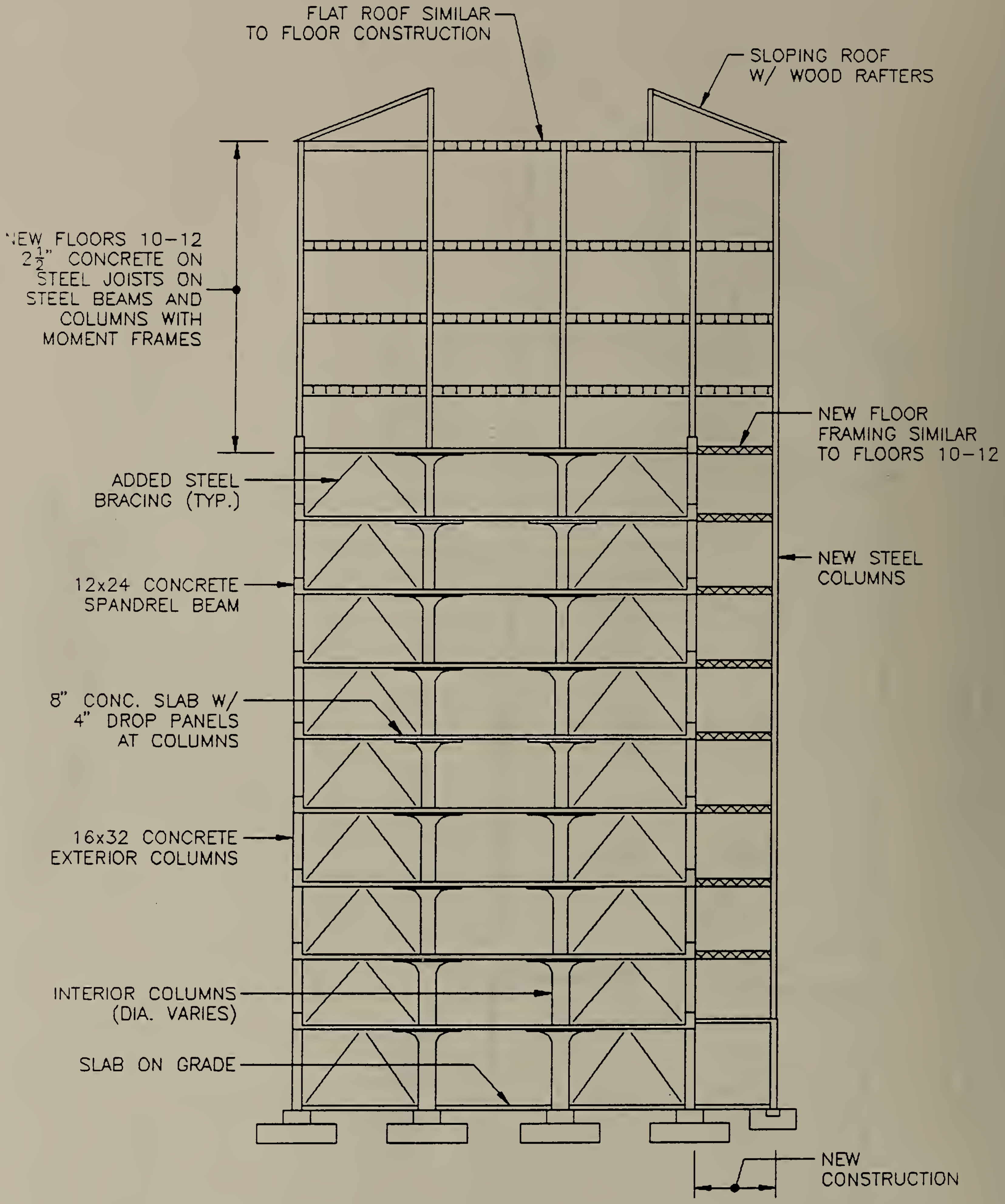
APPENDIX C

BIXBY CROSSING

CALCULATIONS AND ILLUSTRATIONS


CONC. SPANDREL BEAMS
CONCRETE COLUMNS W/ FLARED CAPITALS AND DROP PANELS
60'-0"
LOCATIONS OF ADDED STEEL BRACING (TYP.)
NEW STEEL FRAMED ADDITION
180'-0"

BIXBY CROSSING
TYPICAL FLOOR PLAN



BIXBY CROSSING
BUILDING SECTION

↰ BEFORE

AFTER →

DURING CONSTRUCTION ↴

BOSTON
BUILDING
CONSULTANTS

Subject: BIXBY CROSSING
Job No:
Sheet:
By: ALL
Date: 1/94
Checked:

Floor	DL	Total	Addition
1	130+5	1344	117
2	130+5	1344	53
3	125+5	1296	53
4	125+5	1296	53
5	120+5	1248	53
6	120+5	1248	53
7	115+5	1200	53
8	115+5	1200	53
R(9)	70+5+15	888	269
10			505
11			505
12			505
R			490
Σ		11064	2762

Total W = 13 826 k

Increase in W = 25 %.

Area - original - 9 × 9600 = 86 400
new - 13 × 10800 = 140 400

Increase in area = 63 %

BOSTON
BUILDING
CONSULTANTS

Subject: BIXEY CROSSING
Job No:
Sheet: 2
By: ACx
Date: 1/94
Checked:

Building weight.

Existing construction typical floor
8" slab — 6' sq 4" dia - equiv. 0.4" thickness

columns - 16"-28" ϕ - equiv. 0.5" to 1.8" thickness

total dead load of structure 115 to 130 psf

Roof - 5" slab 70 psf

Add 5psf for misc dead load - services, etc.
15psf - ½ snow load on roof

Exterior walls - assume 10psf -
48 k per floor - new addition 3 k per floor extra
(assume new walls have similar weight to old)

New addition :
2½" concrete on steel joists -

conc	30	
struct.	7	
misc.	5	(inc. partitions)
	42	

new roof - approx 50% is same as floor construction - 42 psf
other half is wood framed - 15 psf
- average total 28 psf

3" conc. topping on old roof - 35 psf

building area - typ. existing floor 160' × 60' : 9600 sf
new addition 100' × 12' : 1200 sf
new floors - 10800 sf

Subject: BIXBY CROSSING
Job No:
Sheet: 3
By: AGS
Date: 1/94
Checked:

Earthquake force from section 3203

Use 2 stage process

Upper steel portion - can use Section 1113 with k = 0.67

W = 2005 k
h_n = 40'
D = 160' 60'
T = 0.16 0.26
C = 0.092 0.078
S = 1.0 (Concrete frame as rigid foundation)
V = 41 k 35 k

For lower concrete part - k = 1.5

W = 11321
h_n = 100'
D = 160' 60'
T = 0.40 0.65
C = 0.063 0.053
S = 1.2
V = 482 k 411 k

ΣV = 523 k 446 k.

Use 93% due to 63% increase in weight

V = 486 k in long direction
415 k in short direction

Consider this to be uniformly distributed to allow simple comparison with wind load stresses obtained from a computer stiffness analysis of the frame.

BOSTON
BUILDING
CONSULTANTS

Subject: BIXBY CROSSING
Job No:
Sheet: 4
By: ACA
Date: 1/94
Checked:

In short direction, total wind load on building is 500 kips carried in four steel braced bays acting compositely with the concrete frame. Seismic load is 415 kips, so wind load will govern, and the solution designed for wind load will meet seismic requirements

In long direction, total wind load on building is 185 kips carried in four concrete frames – it is assumed that the interior frames each carry 1/3 of the load, exterior frames 1/6.

forces from this analysis (carried out in 1986).

typical interior column, bottom story M = 122 k.ft
P = 2 k

Beam/slab element – maximum M = 38 k.ft
(at 4th floor)

for earthquake load of 486 k – these forces become

column M = 320 k.ft
P = 5 k
beam/slab M = 100 k.ft

Consider the lowest column – 28"ϕ
reinf. w/ 12 1 3/8" bars – $A_s = 18.8$ in² (equiv to 12 #11)
$f_c' = 3750$ psi
Spiral ties ½"ϕ smooth @ 3" pitch

gravity load from 400 sf
dead – 9 × (120+5) + 3 × 42 + 30 = 1.28 ksf – 512 k
live – 12 × 50 + 30 = 0.63 ksf – 252 k

Subject: BIXBY CROSSING
Job No:
Sheet: 5
By: ACW
Date: 1/94
Checked:

Ult. gravity load : $1.4 \times 512 + 1.7 \times 252 = 1145$ k
w/ earthquake - add 1.7×5 and multiply by $\frac{3}{4}$ - 865 k.

Ult earthquake moment : $1.1 \times 1.7 \times 320 \times 0.75$: 449 k.ft

Analysis of column capacity using ultimate strength analysis

$\phi : 0.75$ $A_{st} : 18.8$ in²
$f_c' : 3750$ psi $f_y : 33\,000$ psi
$D : 28''$ $A_g : 615.8$ in²
$e : 6.2''$
$p_t : A_{st}/A_g : 0.031$
$m : f_y/0.85 f_c' : 10.35$
$D_s : 23''$

balance conditions with $\varepsilon_c : 0.003$
$\varepsilon_s : 0.0011$
$d_c : 18.5''$ $(0.85 d_c : 15.7'')$
area of conc in compression : 346 in²
$f_c : 3188$ psi

$M : 2\,(9.4 \times 33 \times 11.5) + (308 \times 3.188 \times 6) - (1.7 \times 28 \times 3.188 \times 0.85)$
$= 12\,900$ k in

$\phi M_b : 806$ k.

$P : 3.188\,(308 + 1.7 \times 28) = 1133$ k

$\phi P_b : 850$ k

$e_b : 11.4''$

$e < e_b$ ∴ compression controls,

using the empirical formula

$$P_u = \phi \left[\frac{A_{st} f_y}{\frac{3e}{D_s} + 1} + \frac{A_g f'_c}{\frac{9.6\ De}{(0.8D + 0.67\ D_s)^2} + 1.18} \right]$$

$P_u = 996$ kips > 865 k - ok

Consider beam/slab element.

As no information is available on reinforcement - compare with previous situation.

Original case - dead load 115 psf
live load 100 psf
$w_u = 340$ psf

column strip carries 75% of panel load (15') - $w_u = 5.1$ klf

assuming $M = wl^2/11$ at an interior support - $M = 182$ k.ft
span moment 63 k.ft

For new building with earthquake load, and live load = 50

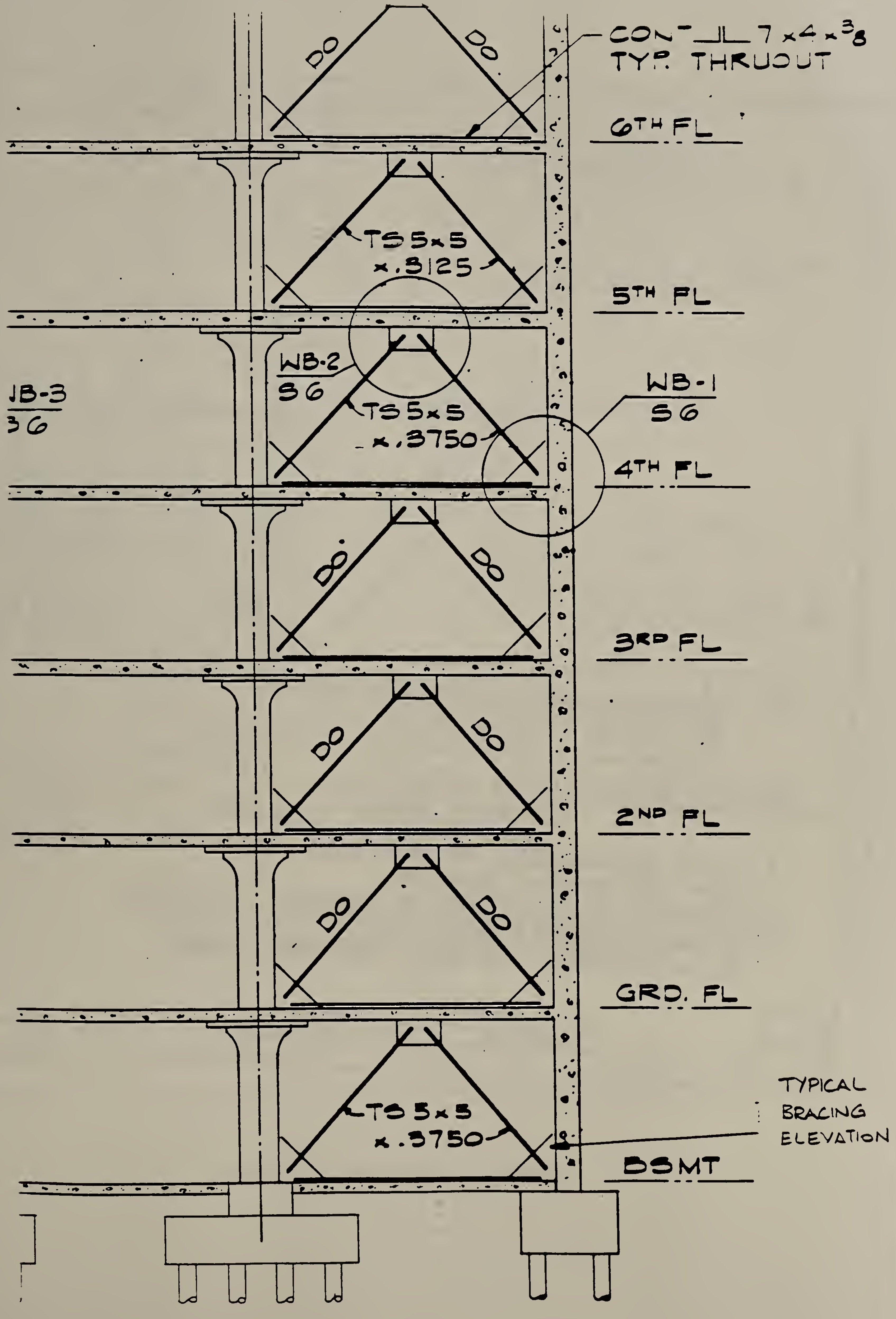
dead + live - $w_u = 245$ psf. - $M_u = 131$ k.ft

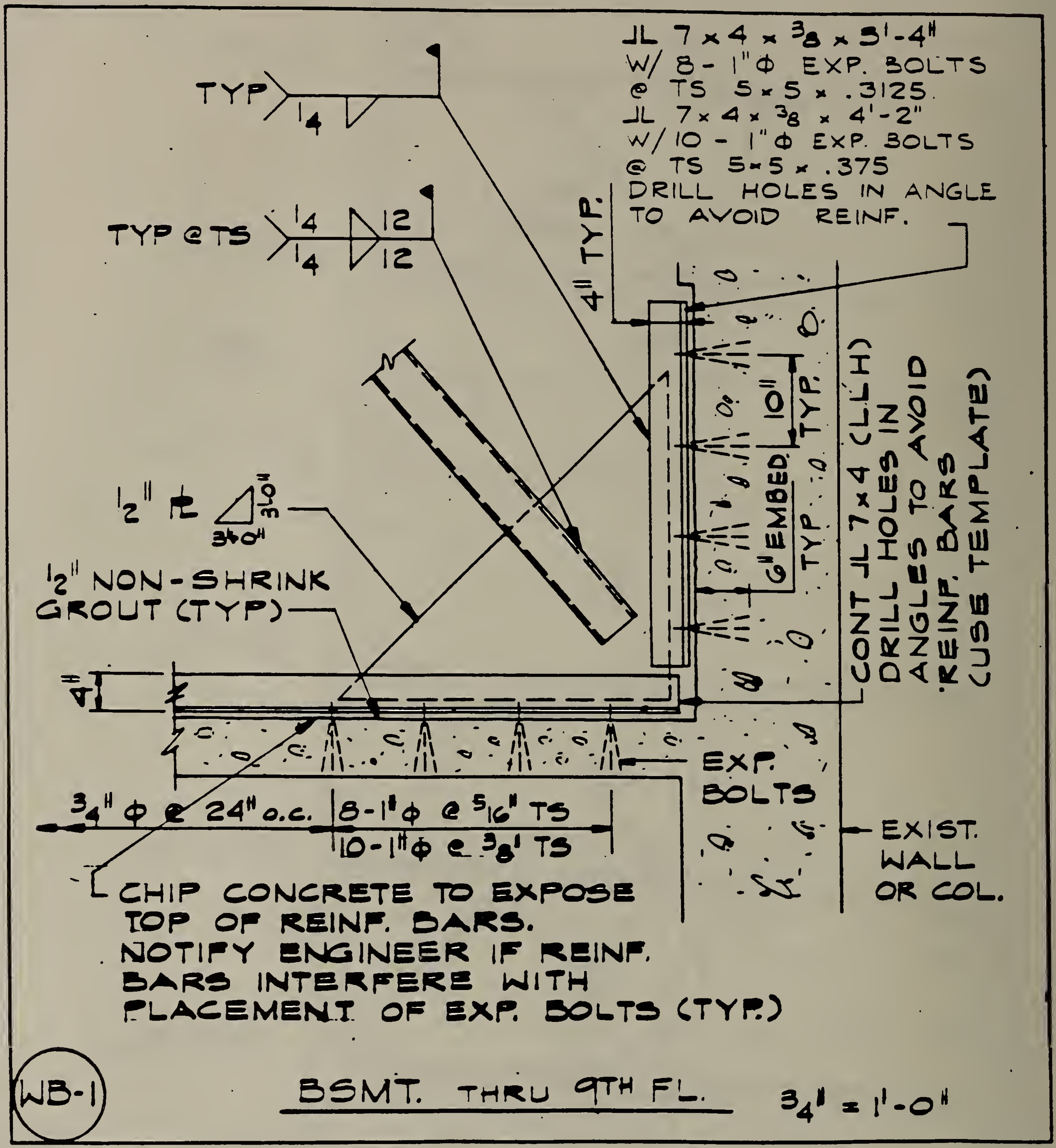
earthquake moment = 100 k.ft

assume 75% carried on column strip $- M_u = 0.75 \times 100 \times 1.1 \times 1.7 = 140$

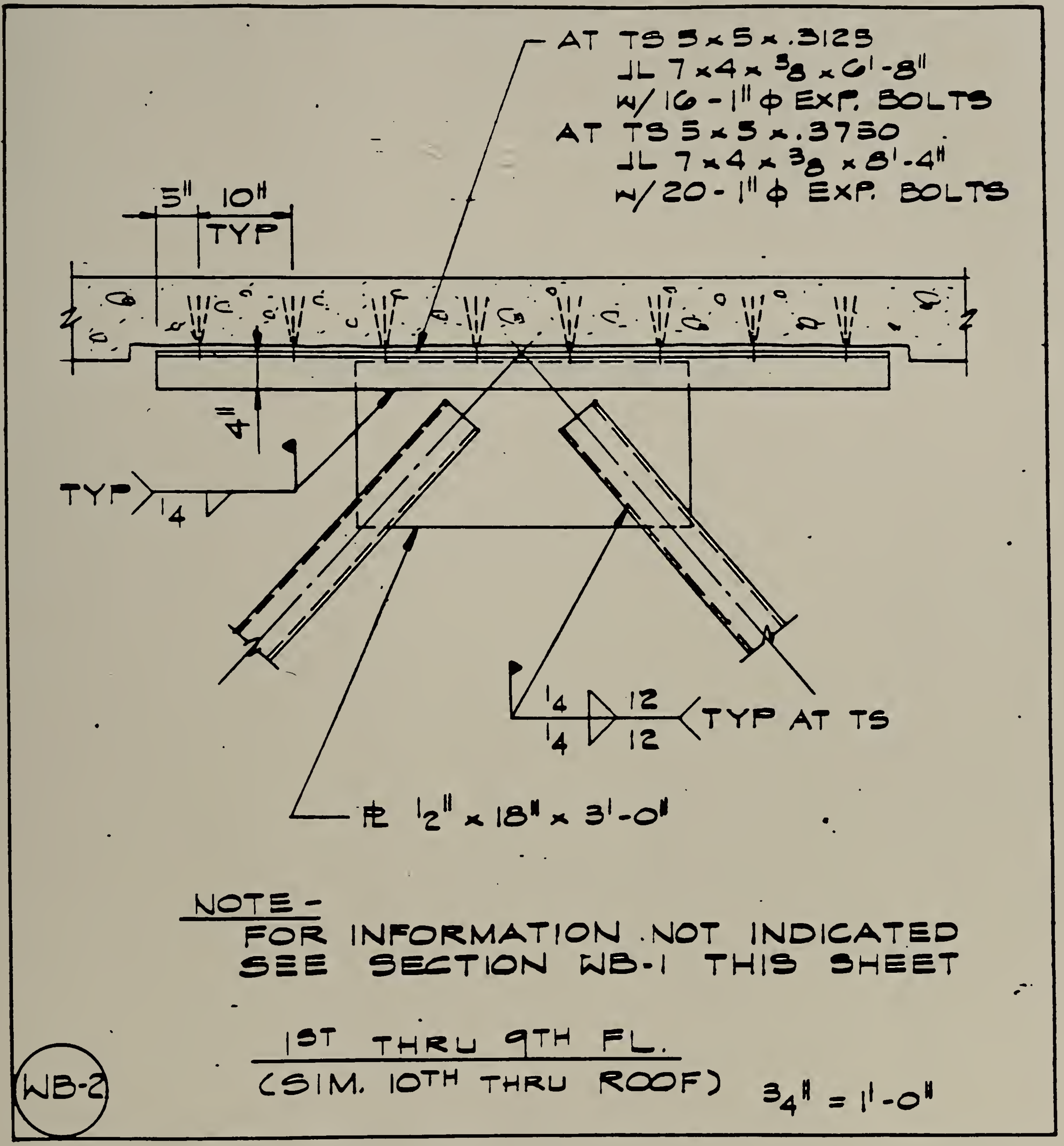
∴ small moment reversal will occur.

Total combined negative moment = 0.75(131 + 140) = 203 k.ft
- an increase of 12% over original de


CONT ⅃L 7x4x3/8
TYP. THRUOUT
6TH FL
DO
DO
TS 5x5
x.3125
5TH FL
WB-2
S6
B-3
6
TS 5x5
x.3750
WB-1
S6
4TH FL
DO
DO
3RD FL
DO
DO
2ND FL
DO
DO
GRD. FL
TS 5x5
x.3750
TYPICAL
BRACING
ELEVATION
BSMT



TYPICAL BRACING DETAIL



NOTE -
FOR INFORMATION NOT INDICATED SEE SECTION WB-1 THIS SHEET

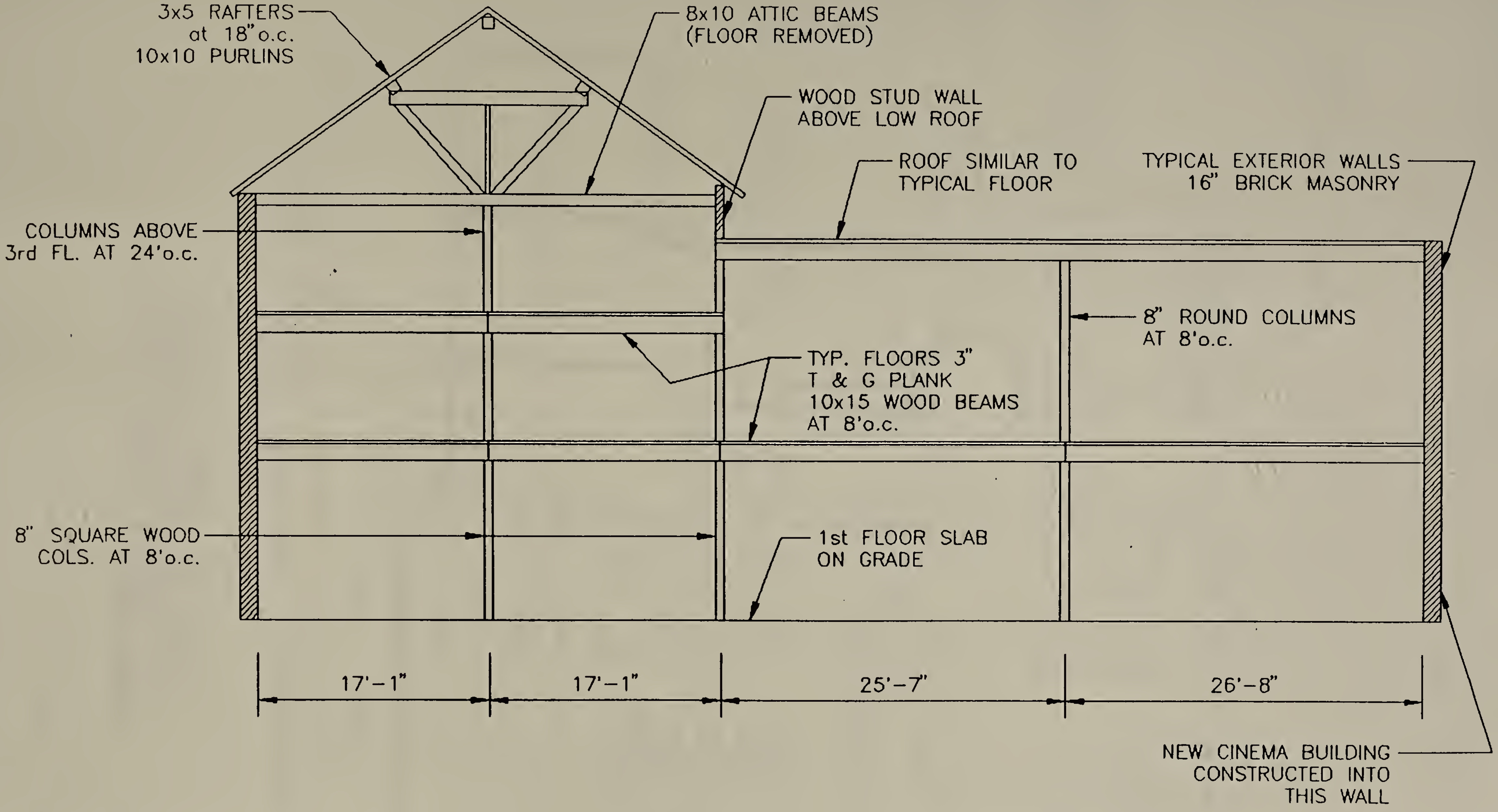
1ST THRU 9TH FL.
(SIM. 10TH THRU ROOF)

TYPICAL BRACING DETAIL

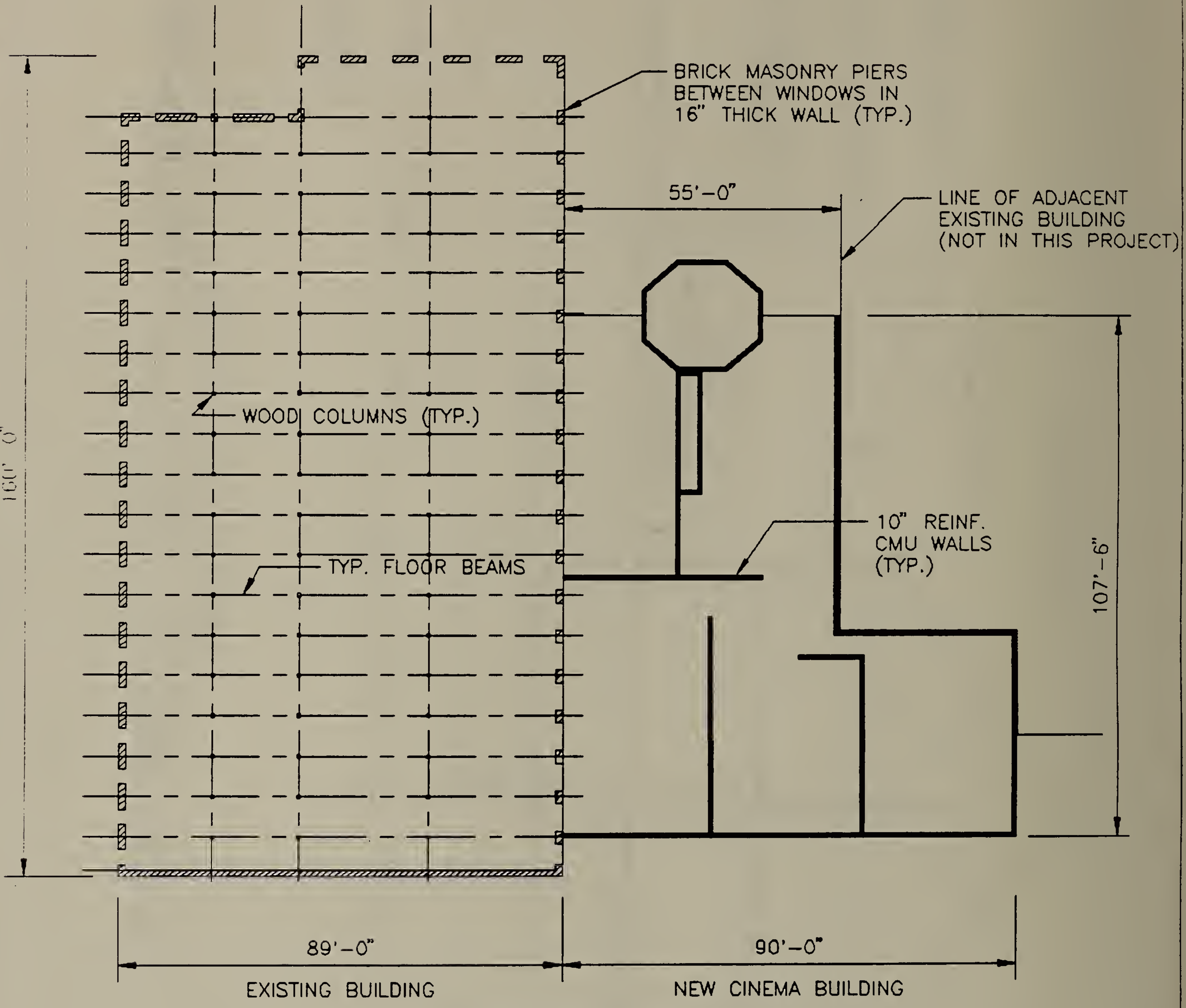
APPENDIX D

LOWELL DYE HOUSE

CALCULATIONS AND ILLUSTRATIONS



LOWELL DYE HOUSE
TYPICAL SECTION THROUGH EXISTING BUILDING


BRICK MASONRY PIERS BETWEEN WINDOWS IN 16" THICK WALL (TYP.)
55'-0"
LINE OF ADJACENT EXISTING BUILDING (NOT IN THIS PROJECT)
WOOD COLUMNS (TYP.)
10" REINF. CMU WALLS (TYP.)
TYP. FLOOR BEAMS
160'-0"
107'-6"
89'-0"
90'-0"
EXISTING BUILDING
NEW CINEMA BUILDING

LOWELL DYE HOUSE SECOND FLOOR PLAN

BOSTON
BUILDING
CONSULTANTS

Subject: LOWELL
DYE
HOUSE

Job No:

Sheet: 2 1

By: ACA Date: 12/93 Checked:

Building weight - existing building

16" brick walls with window openings -
nominal load 160 psf.

on 3 story side - assume 80% solid = 130 psf - 32' × 160'
on 2 story side - assume 50% solid = 80 psf - 28' × 160'
end walls - assume average 80% solid = 130 psf. - 2(34'×38'+5

floor: 3" plank on wood beams and columns = 20 psf
assume roof dead load similar
½ snow load : 15 psf

Floor areas -	2nd floor	86 × 160	:	13 760 sf
	3rd floor	34 × 160	:	5440 sf
	Attic	34 × 160	:	5440 sf
	Roof	86 × 160	:	13 760 sf
		total		38 400 sf

Total weight of existing building :

walls : 32 × 160 × 0.13
28 × 160 × 0.08
(34 × 38 + 53 × 28) × 2 × 0.13
total 1745 k.

floors :	2nd fl	13 760 × 0.02
	3rd fl	5440 × 0.02
	Attic	5440 × 0.02
	Roof	13 760 × 0.035

total 974 k (109 k in attic)

total weight of existing building 2721 [illegible]

Subject: LOWELL DYE HOUSE	Job No:		Sheet D 2
	By:	Date: 12/93	Checked:

building weights & areas - new building

for determination of % increase -
new building is laterally self-supporting.

Roof - steel joists + metal deck - 10 psf
½ snow load 15 psf
area - 54×54 + 44×35 : 4450 sf

6" precast concrete plank : 50 psf
½ snow load 15 psf
area 54×34 : 1840 sf

Mezzanine floor - 6" pc plank w/ 2" topping 65 psf
area - 54×60 + 60×8 : 3770 sf (of which 700 sf is roof

Walls - 10" reinf. CMU - 70 psf area in elevation

506 lf. wall 25' high
45 lf wall 15' high
30 lf wall 18' high
total 13 865 sf

Tower roof - wood framing - 10 psf - area 400 sf

Walls -	13 865 @ 70 psf	970 k
high roof	4450 @ 25	111 k
	1840 @ 65	101 k
tower	400 @ 25	10 k
low roof	700 @ 80	56 k
2nd floor (mezz)	3070 @ 65	200 k
		1448 kips

Total area 10 460 sf

BOSTON
BUILDING
CONSULTANTS

Subject: LOWELL DYE HOUSE

Job No:

Sheet 03

By:

Date: 3/93

Checked:

Areas :

original building	38 400 sf
new building	10 460 sf
remove attic floor	5 440 sf -
	43 420 sf

If attic is considered part of original area
Increase : 43 420/ 38 400 : 13 % increase

If attic is not considered -
Increase : 43420 / 32 960 : 32 % increase

Weights :
No question about inclusion of attic - it is weight !

original building	2721 kips
new building	1448 kips
remove attic	- 109 kips
	4060 kips

Increase : 4060 / 2721 : 50 % increase

Subject: [illegible] DYE HOUSE
Job No:
Sheet [illegible]
By: [illegible]
Date: 12/93
Checked

Analysis of existing masonry building under section 2305.

for masonry bearing wall system - unreinforced - K = 4.0

soil bearing capacity is 4 ksf - so assume S, - S = 1.0

building dimensions : D = 160' , 86'
h_n = 32'

T = 0.13 , 0.17
C = 0.10 , 0.09

C = 0.10 long
C = 0.09 short

building weight - see sheet 01 2721-109 W = 2612 kips

$V = \frac{1}{3} KCSW$

in long direction $V = \frac{1}{3} \times 4 \times 0.1 \times 1 \times 2612$
= 348 kips

in short direction $V = \frac{1}{3} \times 4 \times 0.09 \times 1 \times 2612$
= 313 kips

In long direction - assume half of load to each side, this is conservative because the two story side has less resistance - piers are smaller.

V is resisted by 20 3' long piers of 16" brick

f_v = 15 psi

in short direction - 9 piers totaling 49 lin. ft

f_v = 16 psi

Consider force on walls required to be resisted by ties to floor diaphragm.

Distribution of W by levels. ($F_t = 0$)

	h_x	w_x	$w_x h_x$	F_x/V	F_x
first floor	0	393	0	0	
2nd floor	13	989	12 857	0.27	95
3rd floor	23	376	8648	0.18	64
low roof	28	477	13 356	0.28	98
high roof	32	377	12 064	0.26	91

$\Sigma = 46\,925$

Consider a portion of wall at second floor level on the three-story side of the building.

Wall must span vertically, and be supported by the floors. With the force acting toward the center of the building this could be achieved by direct bearing. In the opposite direction a tie is required.

For forces parallel to the wall - the force must be transferred through the floor diaphragm to the wall - which is a simple bearing force but produces shear in the wood beams. On the end walls.

11.5' height of wall must be supported at second floor - w = 130psf

For a 1' length of wall - $W_p = 1.5$ k

from Section 1113.4 5 for parts of structures

$F_p = \frac{1}{3} C_p W_p$

$C_p = 0.20$ for walls, normal to flat surface

$F_p = 0.1$ klf

However - note 3 applies.

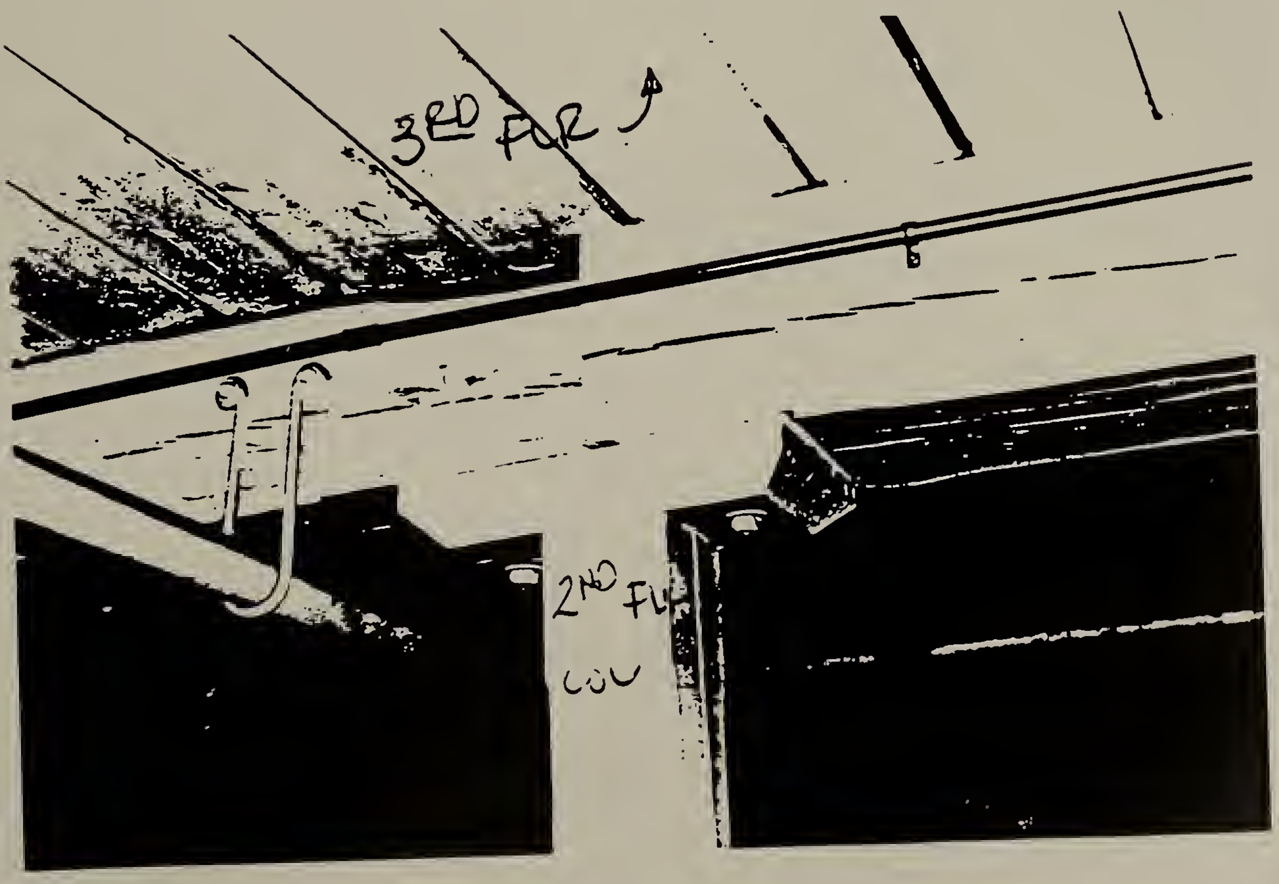
from the force distribution described on sheet D5 - derived from Section 1113.4 2 and $V = \frac{1}{3} KCSW$

force at second floor level is 95k which is equal to $0.27 W_x$

force applied to wall $= 0.27 W_p$
$= 0.4$ klf

(note for $k = 1.33$ - $F = 1.3$ klf)
(Using $V = \frac{1}{3} KCSW$ directly with W_p gives $V = 0.13 W_p = 200$ #/ft)

This appears to require a tie force of 400 # per lin. ft. of wall

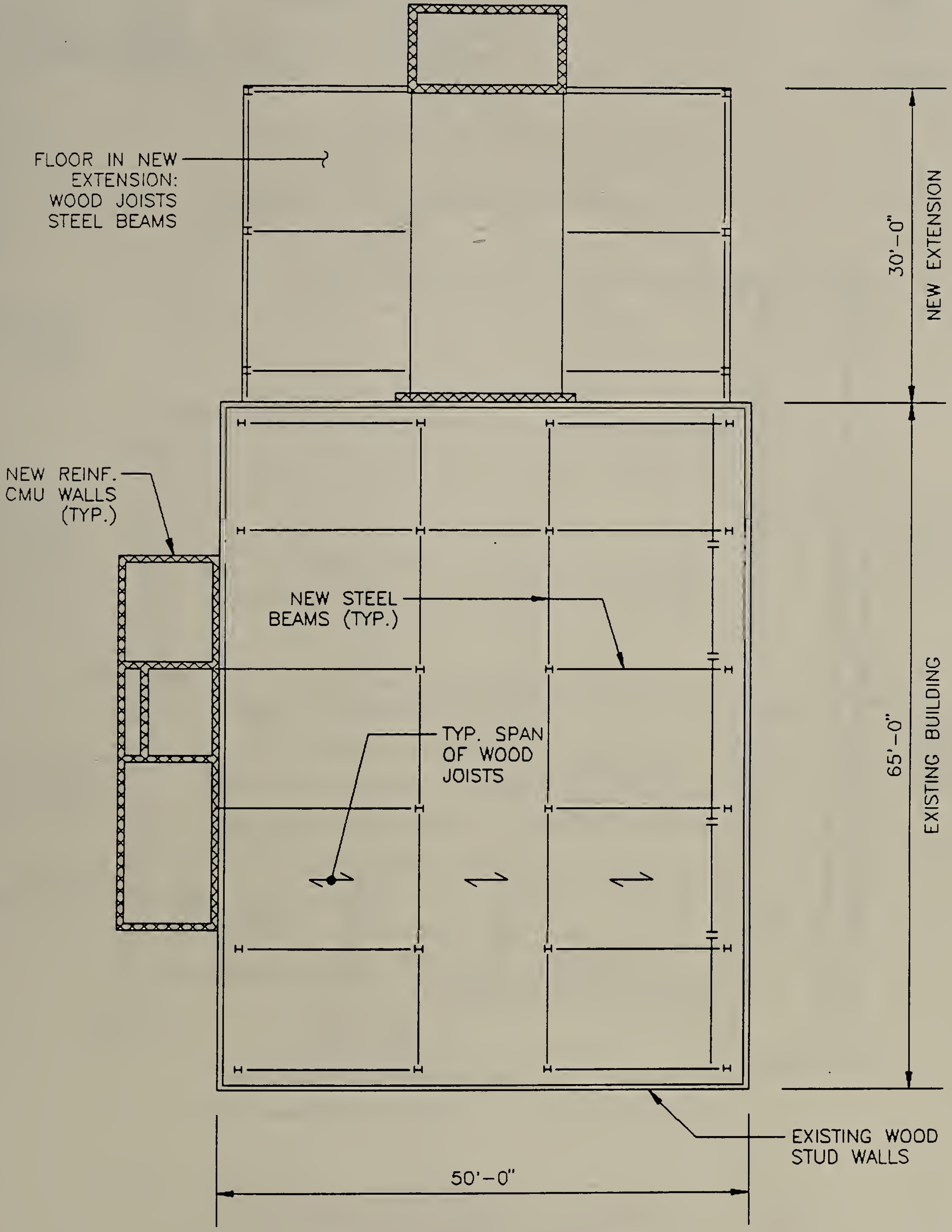

3RD FLR
2ND FL
COL

APPENDIX E

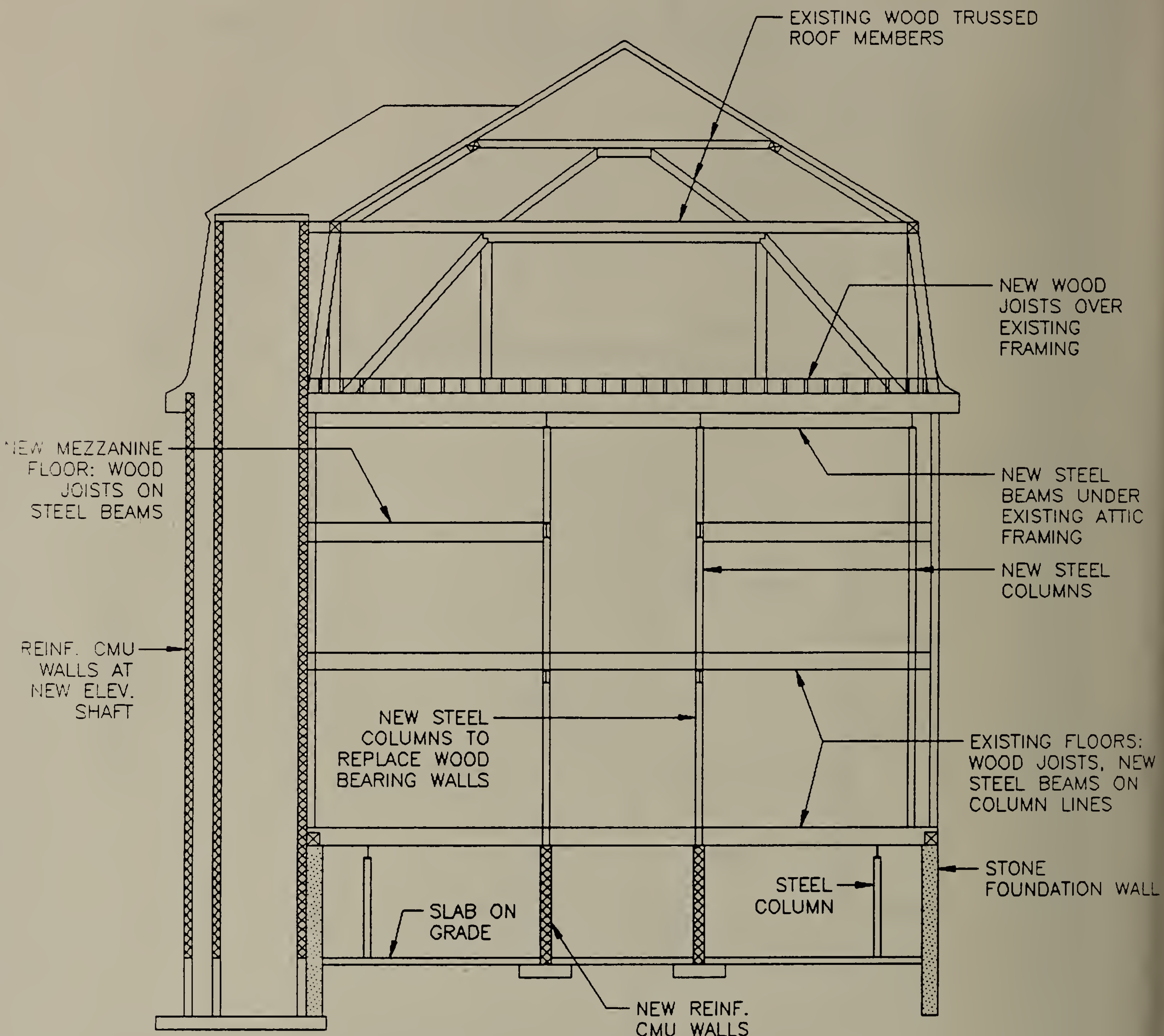
NORTH READING LIBRARY

CALCULATIONS AND ILLUSTRATIONS

FLINT MEMORIAL HALL.
FLINT MEMORIAL LIBRARY



NORTH READING LIBRARY
SECOND FLOOR PLAN


EXISTING WOOD TRUSSED ROOF MEMBERS
NEW WOOD JOISTS OVER EXISTING FRAMING
EW MEZZANINE FLOOR: WOOD JOISTS ON STEEL BEAMS
NEW STEEL BEAMS UNDER EXISTING ATTIC FRAMING
NEW STEEL COLUMNS
REINF. CMU WALLS AT NEW ELEV. SHAFT
NEW STEEL COLUMNS TO REPLACE WOOD BEARING WALLS
EXISTING FLOORS: WOOD JOISTS, NEW STEEL BEAMS ON COLUMN LINES
STONE FOUNDATION WALL
STEEL COLUMN
SLAB ON GRADE
NEW REINF. CMU WALLS

NORTH READING LIBRARY
BUILDING SECTION

BOSTON
BUILDING
CONSULTANTS

Subject: NORTH READING LIBRARY
Job No:
Sheet: 1
By: AGK
Date: 1/94
Checked:

Building weight - (Existing portion only)

Basement :
existing 2 walls × 64' @ 15psf
new replace w/ 10" CMU @ 70psf
} Not req in seismic calcs.

	EXIST		NEW
1st floor			
floor - 3000 sf @ 12 psf	36 k		
14' × 230' exterior wall @ 20psf	64 k		
2 walls × 64' @ 15psf	27 k	removed	-27 k
add ceiling 5psf steel framing 5psf wood 4psf			+42 k
2nd floor			
floor + ceiling 3000 sf @ 20psf	60 k		
10' × 230' exterior wall @ 20psf	46 k		
light partition say 5psf on plan	15 k	removed	-15 k
add steel framing 3psf wood 4psf			+21 k
Mezzanine			
10' × 230' exterior wall @ 20psf	46 k		
add new floor 2500 sf @ 15psf			+38 k
3rd floor			
floor + ceiling 3000 sf @ 20psf	60 k		
add wood floor 3000 sf @ 7 psf steel framing 3psf			+ 30 k
Roof			
mansard - 230' × 14' @ 20 psf	64 k		
top 2500 sf @ 20 psf + 15psf snow	88 k.		
	506 k		+89 k

Total new weight 595 k

BOSTON
BUILDING
CONSULTANTS

Subject: NORTH READING LIBRARY

Job No:

Sheet: 2

By: AGJ Date: 1/94 Checked:

North addition :

- 8" solid grouted CMU - 75psf
40' high × 110 lin.ft.
+ 10' × 30'
total 4700 sf - 353 k

floor 4 × 80 × 15psf 5 k
stair 4 × 100 × 20psf 8 k
new roof 35×9 × (20+15) 11 k

377 k

East addition.

floors - 6psf steel 20psf total
1350 sf × 2 floors 54 k
roof 15+15psf 40 k
exterior wall 90'×26' @ 20psf 47 k
CMU walls 60'×26' @ 90psf (10") 140 k

281 k

Summary of weights.

Original building	506 k	
Renovated original	595 k	+ 18 %
Total renovated + extended	1253 k	+ 150 %

Subject: NORTH READING LIBRARY
Job No:
Sheet: 3
By: ACK
Date: 1/94
Checked:

Earthquake loading

in N-S direction — E-W

$D = 60'$ — $D = 100'$

$h_n = 50'$

$T = 0.32$ — $T = 0.25$

$C = 0.073$ — $C = 0.079$

$S = 1$

$K = 1.33$

$W = 1253\ k$

$V = \frac{1}{3} KCSW = 41\ k,\ 44\ k$

Less than wind load –

Wind load governs

BOSTON
BUILDING
CONSULTANTS

Subject: N. READING LIB.

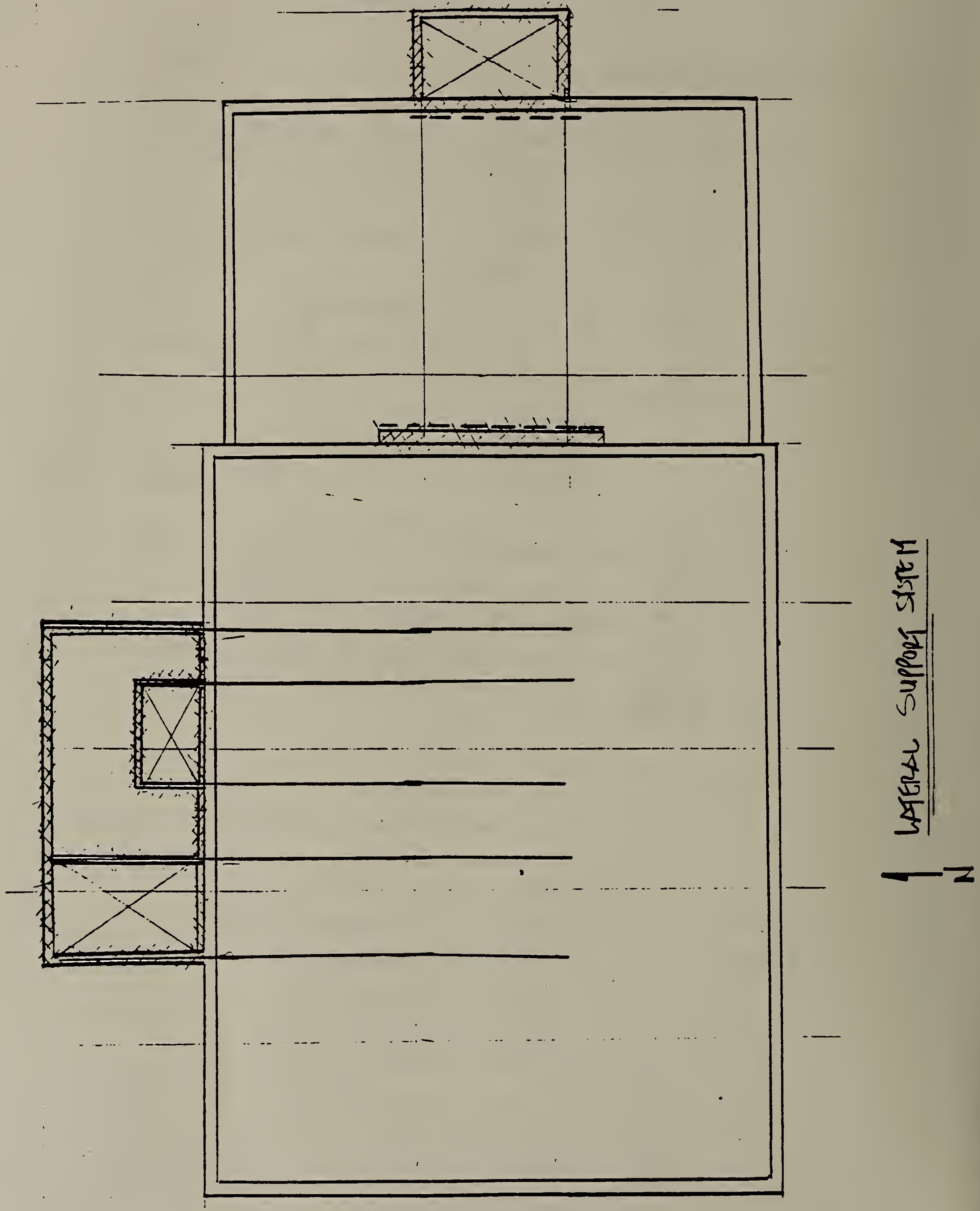
Job No: 800091

Sheet:

By: OP

Date: 2-7-90

Checked:



BOSTON
BUILDING
CONSULTANTS

Subject: N. READING HS

Job No: 88091

Sheet:

By: DP

Date: 2-6-90

Checked:

LATERAL ANALYSIS

EXIST. BLDG

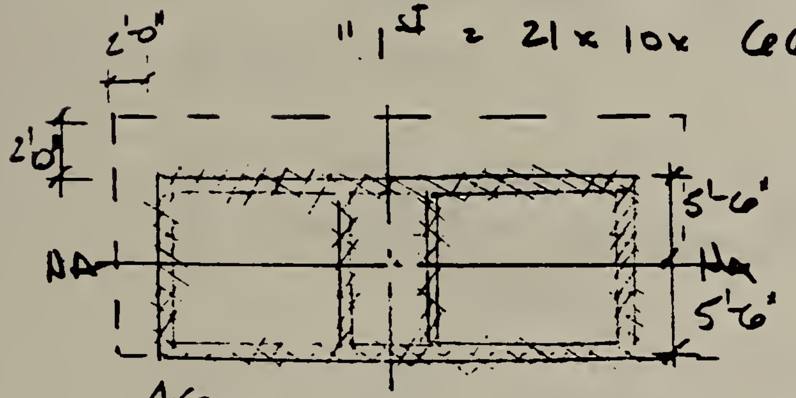
IN N-S DIRECTION EXIST BLDG TO BE SUPPORTED BY NEW ELEV/ STAIR CORE AT NORTH WALL

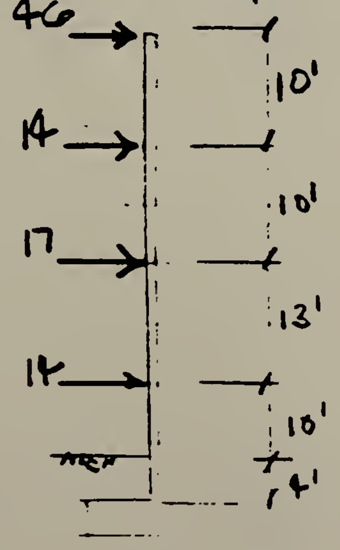
WIND FROM SOUTH

HORIZ. LOAD TO FLR FLEXIBLE DIAPHRAGM. TRANSFERRED TO CMU BY TRIBUTARY AREAS

1) CHECK OVERTURNING:

WIND = 21#/□
AT RF = 21 × 33' × 66' = 46
" 3RD = 21 × 10 × 66 = 14
" 2ND = 21 × 12 × 66 = 17
" 1ST = 21 × 10 × 66 = 14 / 91K





$M_O = 46(47') + 14(37) + 17(27) + 14(14) = 3335^{\prime K}$

DL
WALL 6" SOLID = 75#/□ × [(11 × 4 + 35 × 2)] × 60 = 513K
FLR DL = 12' × 9' × 20#/□ × 4 FL = 9
RF DL = 34' × 11' × 20 = 7
FTG = 38' × 13' × 300#/□ = 148
677K

$M_R = (677)(7.5) = 5078$ $M_R/M_O = 1.52 > 1.5$ OK

Subject: N. REGOING UG

Job No: 88091

Sheet:

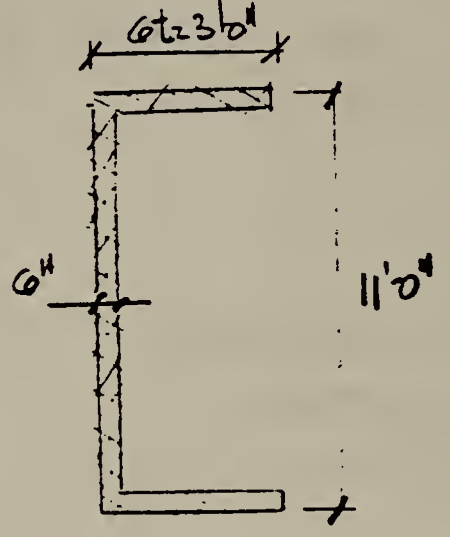
By: OP

Date: 2-6-90

Checked:

CHECK STRESSES

ISOLATE WALL SECTIONS



$A_{FL} = 3(6") = 1.5 FT^2$

$A_{WEB} = 10(0.5) = 5.0 FT^2$

$\bar{x} = 5'-6"$

$I_x = 3(11^3-10^3)/12 = 5.25$

$0.5(10)^3/12 = 41.67 \,/\, 46.92 FT^4$

$S_x = 46.92/5.5 = 8.53 FT^3$

SHEAR

END WALL TAKES EXTRA TRIB AREA DUE TO THAT FACE

TRIB = 13.2 + 6.6 = 19.8 % = 19.8/64 = 30%

SHEAR = $91^k \times 0.3 = 27^k$

STRESS = V = $27/6.5 \times 144$ = 29 psi < 24 psi allowable INSPECTED

CHK FOR TENSION

DL + WIND

$f_m = \frac{P}{A} + M/S \quad P = 677 - 148 = 529^k$

$= \frac{529}{6.5(144)} \pm \frac{3335(.3)}{8.53(144)}$

$= 565 \pm 814 = 1380$ psi comp

250 psi TENSION

BOSTON
BUILDING
CONSULTANTS

Subject: N BEARING

Job No: 38091

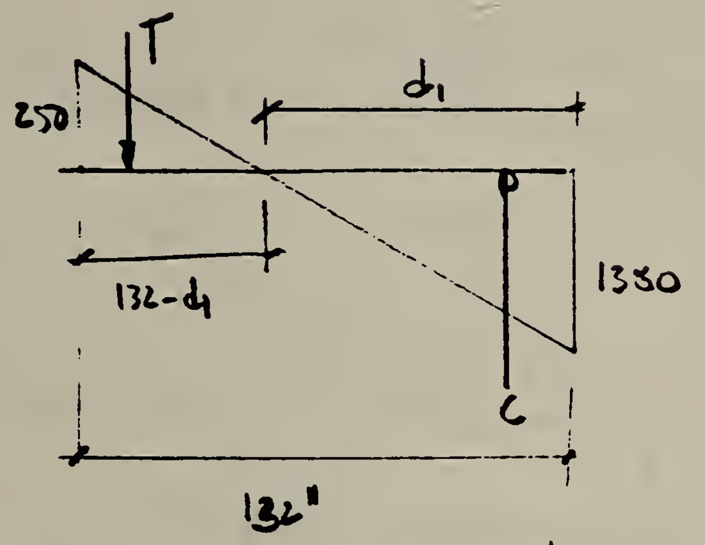
Sheet:

By: α

Date: 2-7-90

Checked:

STEEL REQMTS



$\frac{1380}{d_1} = \frac{1380+250}{132}$ $d_1 = 112"$

$132 - d = 20"$

$T = 1/2(250)(20)(6") = 15^k$ $A_s = 15/60 = .25$ 2-#5 at $A_s = .62\ in^2$

$C = 1/2(1380)(112)(6) = 464^k$

CHK. COMBINED STRESS

DL+LL+ WIND

$\frac{f_a}{F_a} = \frac{529/6.x(144)}{.188 f'_m} = 3/f'_m$ Σ

$\frac{f_b}{F_b} = \frac{314}{0.3 f'_m} = 2.713/f'_m$

$\frac{5.716}{f'_m} = 1.33$ $f'_m = 4300$ OK USE

$F_a = 4300(.188) = 808$ psi   use f_m = [illegible]

$F_b = 4300(.3) = 1290$ psi

$f_a = 565 < F_a$ ok   THIS INCL. MCB @ 3' CMU for

f_b = [illegible] < F_b ok

Subject: N. REED

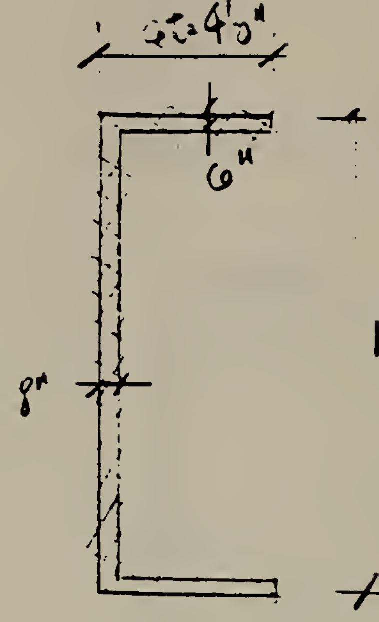
Job No: 58091

Sheet:

By: BP

Date: 2-7-90

Checked:



$A_{FL} = 4(0.5) = 2$

$A_{WEB} = 11(.67) = 7.33 / 9.33\ FT^2$

$\bar{x} = 5'6"$

$I_x = 4(11^3 - 10^3)/12 = 7$

$0.67(11)^3/12 = 74 / 81\ FT^4$

$S_x = 81/5.5 = 14.74\ FT^3$

SHEAR

$v = 27^k / 9.33(144) = 20$ psi ok

CHK FOR TENSION

DL + WIND

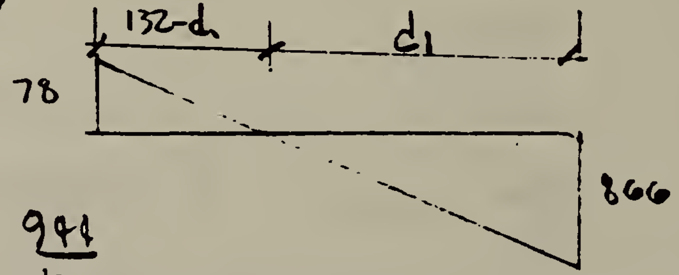
$f_a = P = 529^k \quad P/A = 394$

$f_b = M = 3335^{'k} \quad M/S = 1.57 \times 0.3 = 472$

$f_A = 394 + 472 = 866$ psi COMP

$= 394 - 472 = 78$ psi TENSION

STEEL TENSION



$\frac{866}{d_1} = \frac{944}{132}$

$d_1 = 121 \qquad 132 - d_1 = 11"$

$T = \frac{1}{2}(78)(11)(8") = 3.4^k$

$C = \frac{1}{2}(866)(121)(8) = 419^k$

$A_s = \frac{I}{f_y} = 3.4/40 = 0.085$

USE 2-#5

B O S T O N
BUILDING
CONSULTANTS

Subject: N. READ UB

Job No: 88091

Sheet:

By: MP

Date: 2-7-90

Checked:

CHK. COMBINED STRESS

$$\frac{f_a}{F_a} = \frac{.394}{.188 f'_m} = \frac{2.10}{f'_m}$$

$$\frac{f_b}{F_b} = \frac{.472}{.3 f'_m} = \frac{1.57}{f'_m}$$

$$> \frac{3.67}{f'_m} = 1.33$$

$f'_m = 2760$ USE 3000 psi CMU

$F_a = .188(2760) = 519$ psi $> f_a$

$F_b = .3(2760) = 828$ psi $> f_b$

BOSTON BUILDING CONSULTANTS

Subject: N, 2560 WB

Job No: 88001

Sheet:

By: OP

Date: 12-29-89

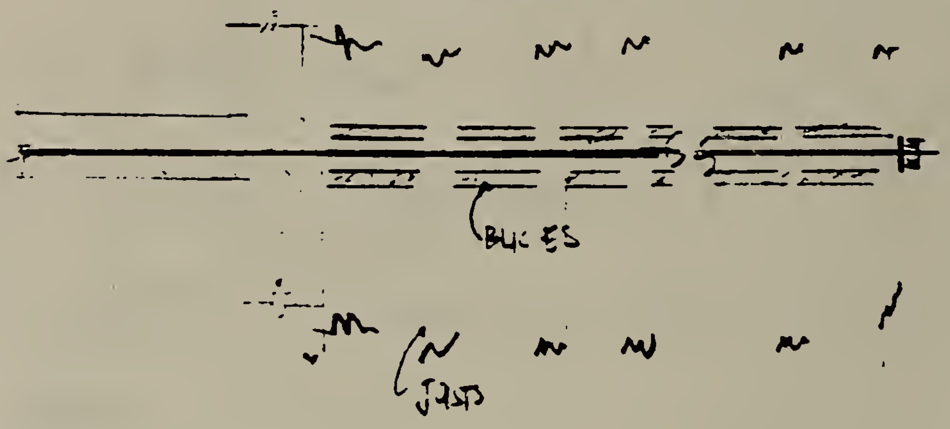
Checked:

WIND FROM THE NORTH

LOAD INTO CHU BY TIE RODS IN TENSION:

WD = 700#/1 × 32' (NORTHSIDE EXCLUDING CHU TOWER) = 22.4K

DISTRIBUTE EQUALLY TO 4 RODS = 5.7K/ROD



NAILS IN BLKING TO TRANSFER LOAD TO ROD:

NAIL WD SHR ~ 50#

NAILS @ 6" OC = 100#/1

2 SIDES = 200#/1

∴ LENGTH OF ROD = 5.7/.2 = 28.7'

SAY 30'0"

– PLACE PLY OVER ROD TO FORM DIAPHRAGM ACTION

WIND FROM WEST

WD = 21#/□ × 50' × 60' = 63K / 2 WALLS = 31.5K/WALL

LENGTH (USE (F) & (G)) = 68 + 32 = 100'

SHEAR = 315#/1 OK

ADD PLY TO INSIDE FACE OF EXIST NAIL @ 6" OC 1/2" PLY 10d NAIL @ 6" OC ALL EDGES BLKED

WIND FROM THE NORTH @ LOWER FLRS

WD = 21#/□ × 11' × 41' = 9.5K / 5 RODS = 1.9K

LENGTH = 1.9/.2 = 10'0" SAY 12'0" TO STEEL BMS.

BOSTON
BUILDING
CONSULTANTS

Subject: N. READ LIB

Job No: 88091

Sheet:

By: DP

Date: 2-7.90

Checked:

LATERAL ANALYSIS

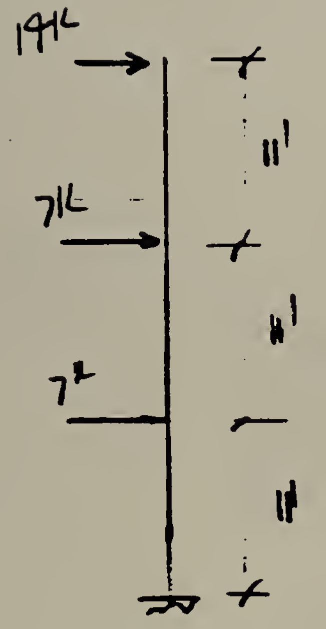
NEW WING - DESIGN

EW DIRECTION SUPPORTED BY PLY SHEAR WALLS
IN ACCORDANCE WITH PREVIOUS CALCS

NS DIRECTION SUPPORTED BY CMU

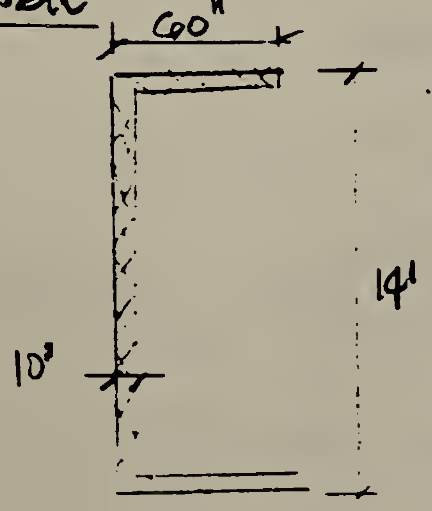
WIND LOAD:

RF = 22 × 21 × 30 = 14^K
2^ND = 11 × 21 × 30 = 7
1^ST = 11 × 21 × 30 = 7
28^K



$M_0 = 7(11) + 7(22) + 14(33) = 700^{\text{'K}}$

EAST WALL



$A_{FL} = 5(10/12)\,2 = 8.33$
$A_{WEB} = (14 - 20/12)(10/12) = 10.3 \;/\; 18.6\ \text{FT}^2$

$I_x = 5(14^3 - 12.33^3)/12 = 362$
$\quad 10/12(12.33)^3/12 = 130 \;/\; 492\ \text{FT}^4$

$S_x = 492/7 = 70\ \text{FT}^3$

BOSTON
BUILDING
CONSULTANTS

Subject: N. READ LIB.

Job No: 88091

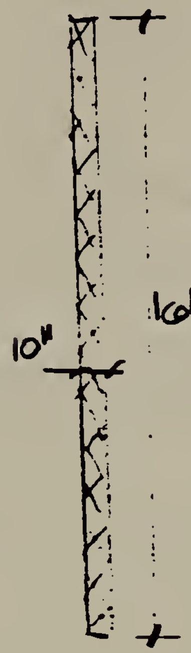
Sheet:

By: DP

Date: 2-7-90

Checked:

WEST WALL



AREA = $16(10/12) = 13.33\ FT^2$

$I_x = \frac{10}{12}(16)^3/12 = 284\ FT^4$

$S_x = 284/8 = 35.6\ FT^3$

% of TOTAL $\frac{I_x}{\Sigma I} = 284/(284+492) = 37\%$

∴ EAST WALL = 63%

STRESS ANALYSIS

EAST WALL

SHEAR = $28^k(.63)/18.6 \times 144 = 7$ psi OK

M_R

$DL = \frac{10}{12}(150)(14\times2+8\times2)(33) = 182^k$ WALL

$150(1)(16\times10) = \underline{24^k}$ FTG

206^k

$M_R = 206(8') = 1650^{'k}$

$M_R/M_O = 1650/700 \times .63 = 3.7$ OK

CHK FOR TENSION DL + WIND

$P/A = 182^k/18.6 = 68$ psi

$M/S = \frac{700(.63)}{70(144)} = 44$ psi

MAX = 112 psi

MIN = 24 psi NO TENSION.

Subject:

Job No: 88091

Sheet:

By: DP

Date: 2-7-90

Checked:

N. LEAD WB

CHK COMBINED STRESS
DL+LL +WIND

LOADS

CMU WALL = 182^k
PLR .41^k × 2 = 82^k
RF .60 × 15 × 15 = 14
NEWEL = 30 × 30 × 15 = 14
292^k

$$\frac{f_a}{F_a} = \frac{292^k/18.6\times144}{.188\,f'_m} = \frac{.58}{f'_m}$$

$$\frac{f_b}{F_b} = \frac{.044}{.3\,f'_m} = \frac{.15}{f'_m}$$

$$\frac{.73}{f'_m} = 1.33$$

f'_m = 550 psi OK

NORMAL 2000 psi OK

WEST WALL

SHEAR = 28(.37)/13.33(144) = 5 psi OK

Mo = 700 × .37 = 260^{1k}

Mr DL: WALL = $\frac{10}{12}$(150)(13.33)(33) = 55^k
PLR = 20 (15 × 15) × 2 = 9
64^k
FTG = .150(3)(18) = 8
72^k

Mr = 72^k × 8 = 583^{1k}

Mr/Mo = 2.2 OK

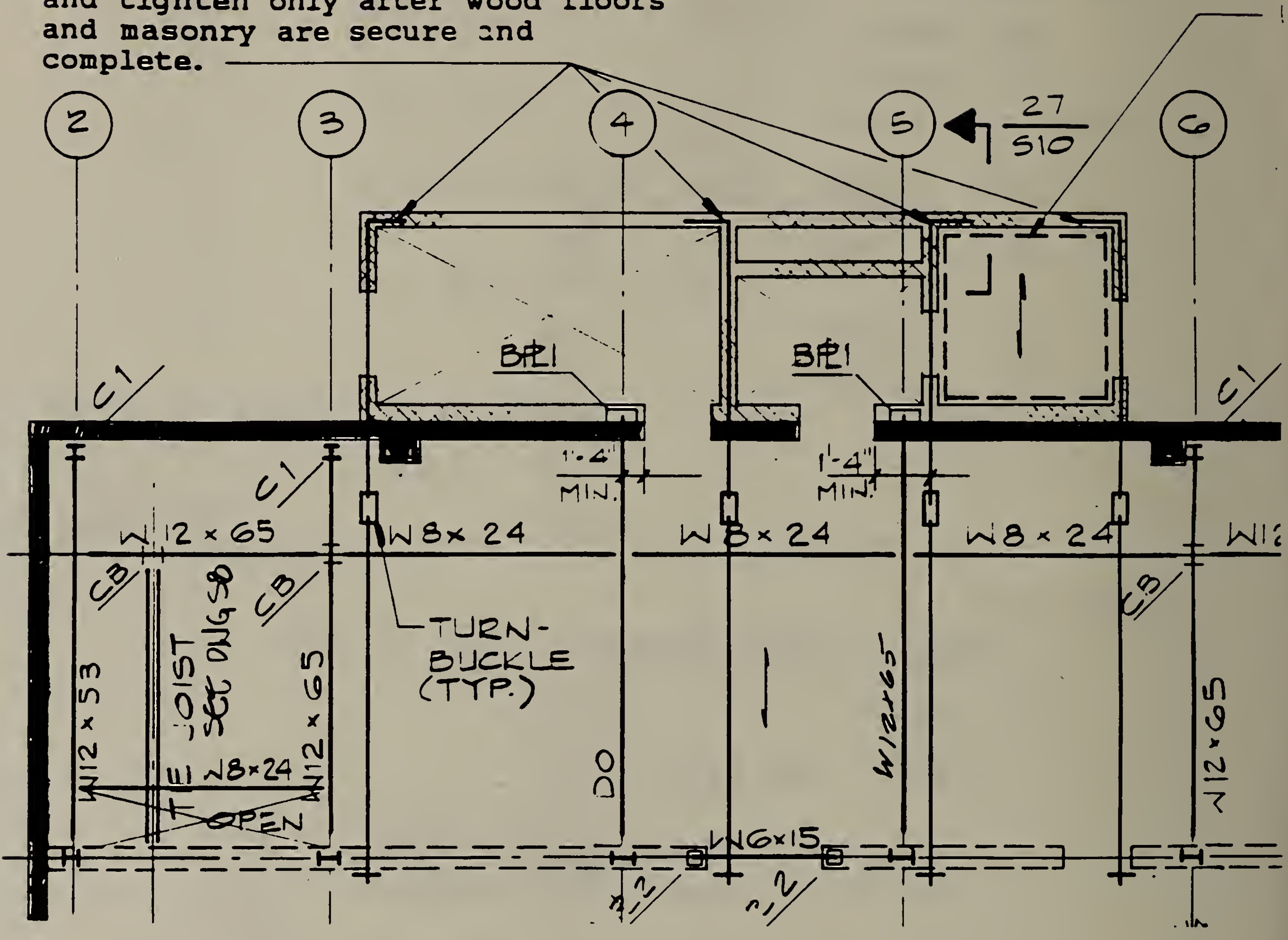
CHK FOR TENSION (DL+WIND)

P/A = 64/13.33(144) = 33 psi   MAX = 84 psi

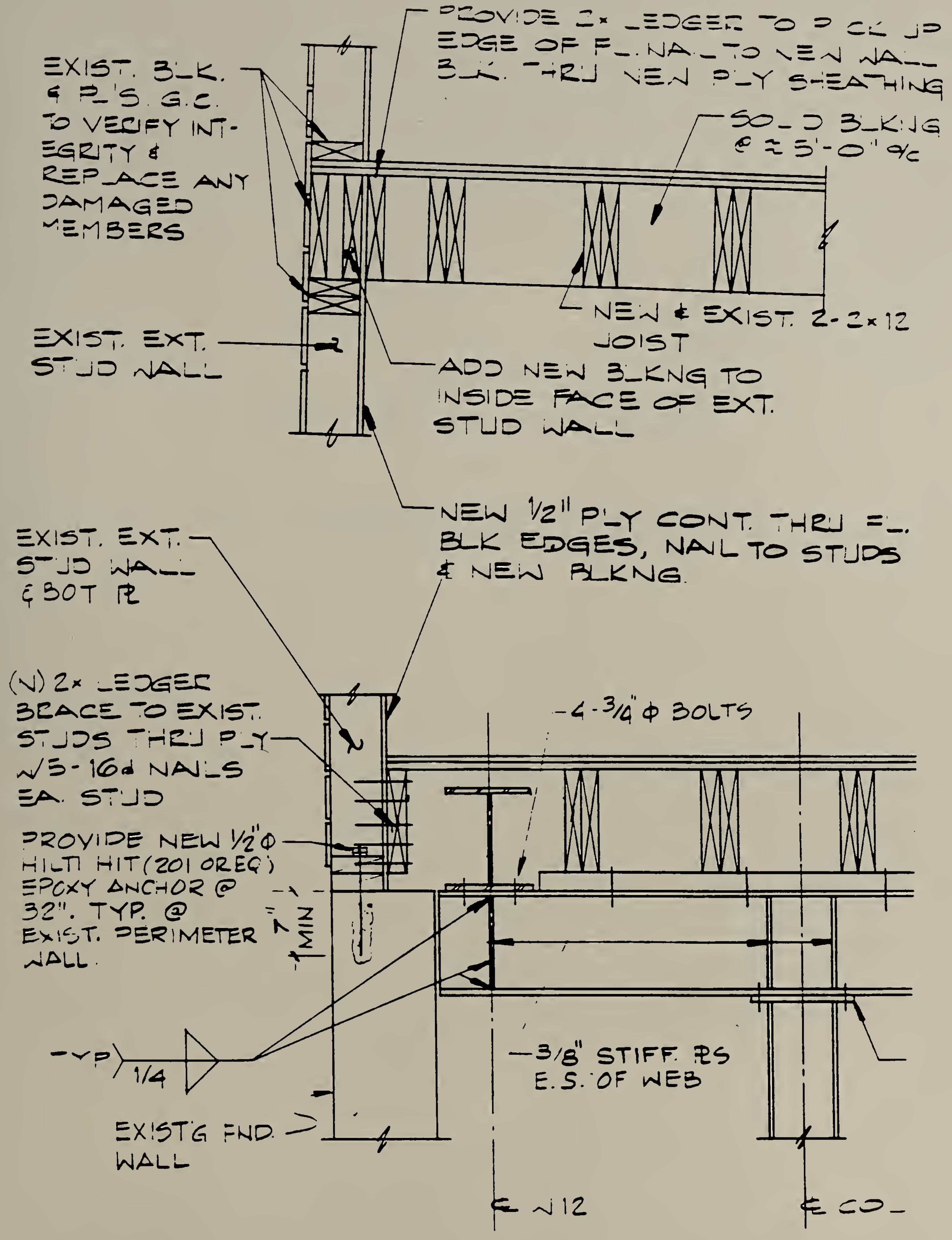
M/S = 260^{1k}/35.6 × 144 = 51 psi   MIN = -18 psi Tension

1-#5 OK

1" diameter threaded rod from new masonry structure to interior of existing wood structure as shown. Embed rod in CMU bond beam and extend rod 2'-0" around CMU corner. Provide solid blocking or one floor joist each side of rod in wood structure and nail to flooring with 10d nails at 4" o.c. Provide rod turnbuckle in wood structure and tighten only after wood floors and masonry are secure and complete.



PART PLAN - FLOOR TIES


PROVIDE 2x LEDGER TO PICK UP EDGE OF FL. NAIL TO NEW WALL BLK. THRU NEW PLY SHEATHING
EXIST. BLK. & PL'S. G.C. TO VERIFY INTEGRITY & REPLACE ANY DAMAGED MEMBERS
SOLID BLKING @ ±5'-0" o/c
NEW & EXIST. 2-2x12 JOIST
EXIST. EXT. STUD WALL
ADD NEW BLKNG TO INSIDE FACE OF EXT. STUD WALL
NEW 1/2" PLY CONT. THRU FL. BLK EDGES, NAIL TO STUDS & NEW BLKNG.
EXIST. EXT. STUD WALL & BOT PL
(N) 2x LEDGER BRACE TO EXIST. STUDS THRU PLY W/5-16d NAILS EA. STUD
4-3/4" Φ BOLTS
PROVIDE NEW 1/2"Φ HILTI HIT (201 OR EQ) EPOXY ANCHOR @ 32". TYP. @ EXIST. PERIMETER WALL
7" MIN
TYP
1/4
3/8" STIFF PLS E.S. OF WEB
EXIST'G FND. WALL
℄ W12
℄ COL

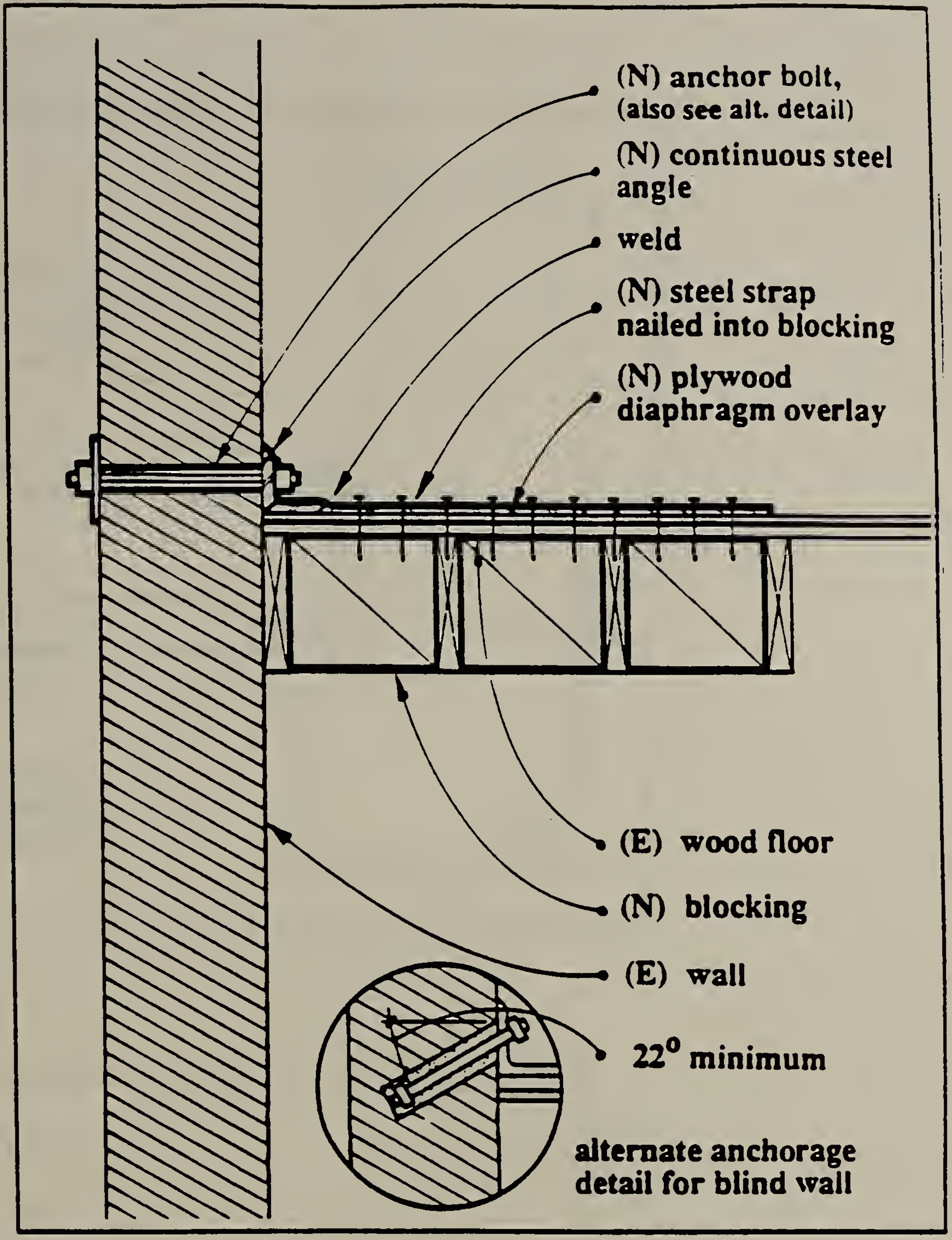
TYPICAL DETAILS
WALL SHEATHING

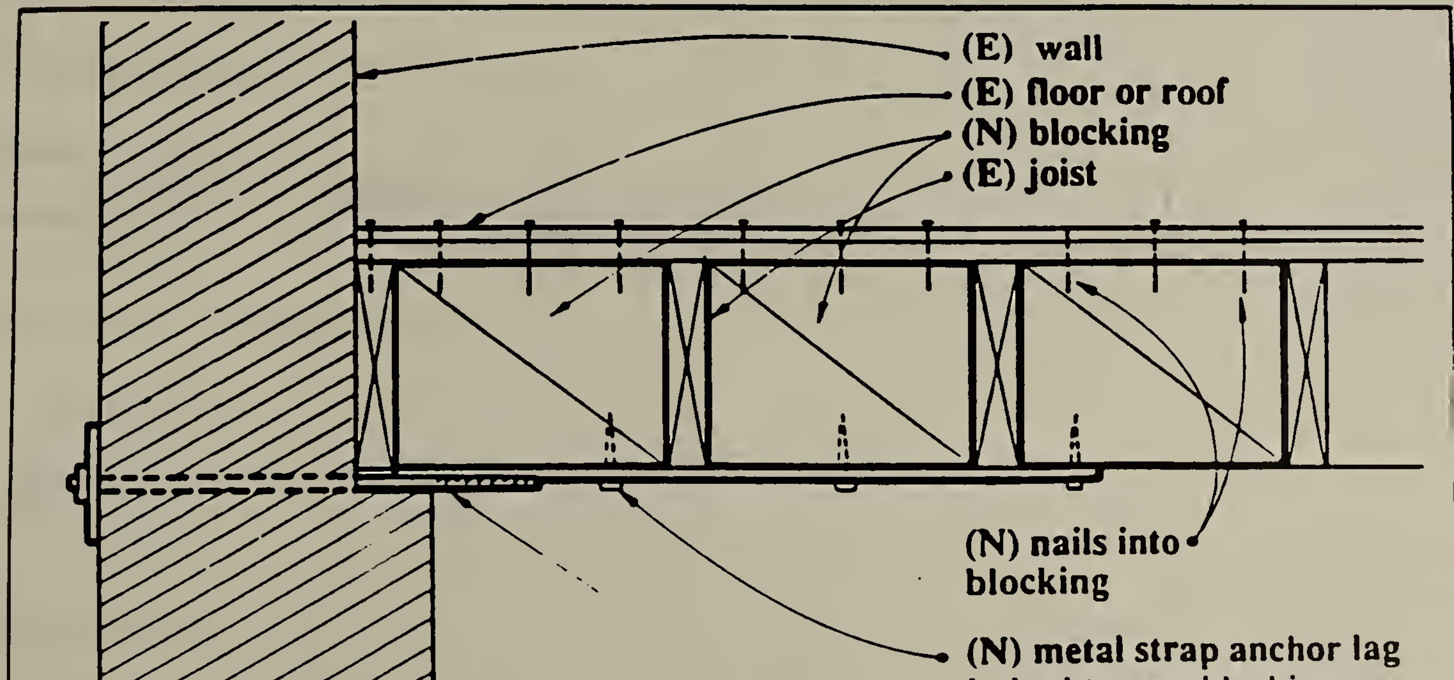
APPENDIX F

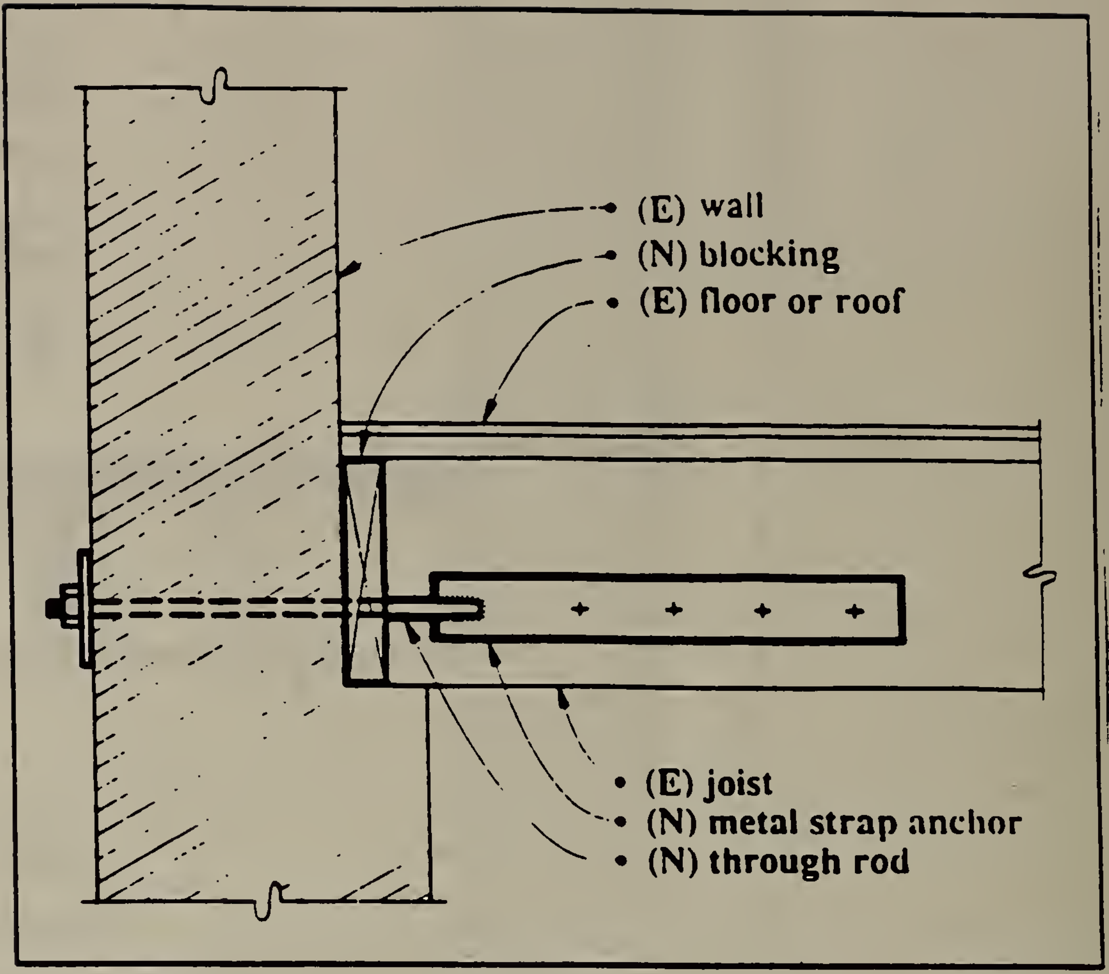
FLOOR TO WALL TIE TYPICAL DETAILS

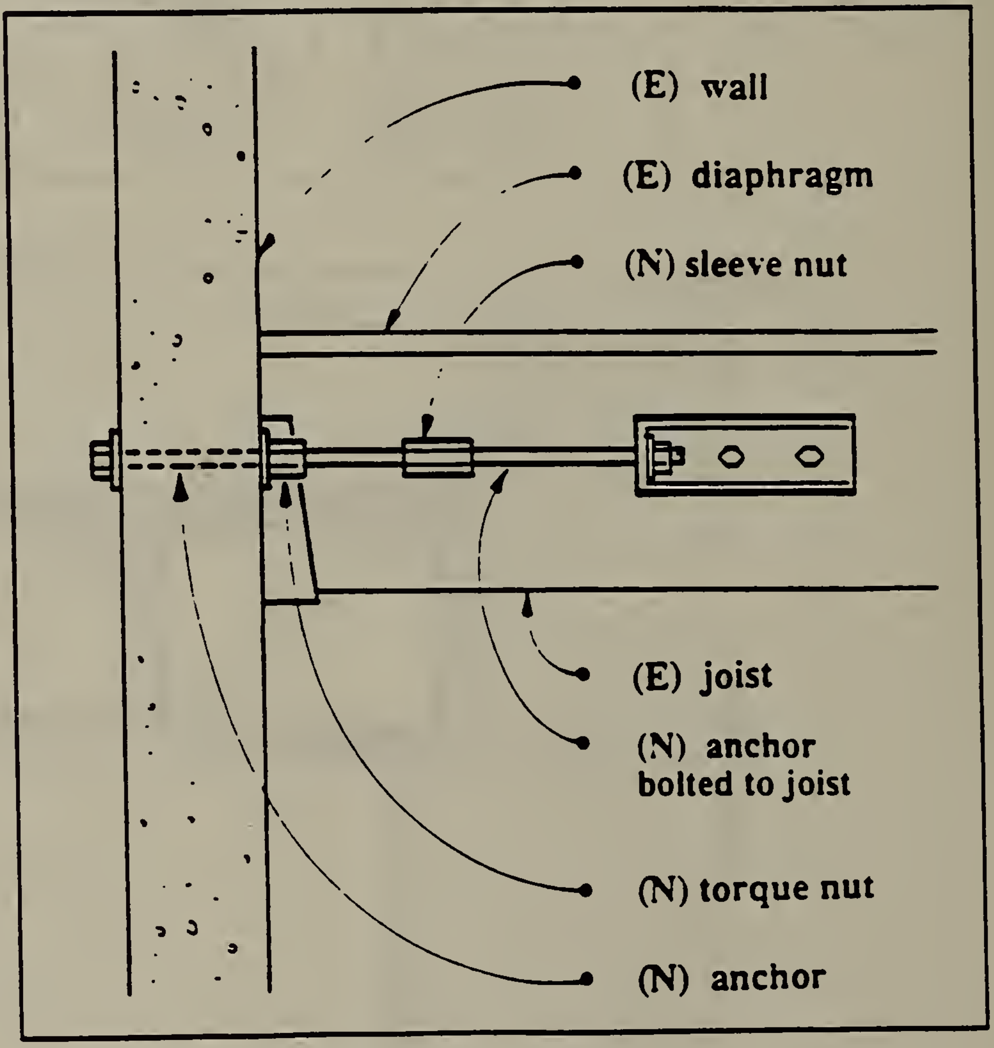
from

NEHRP HANDBOOK FOR SEISMIC REHABILITATION
OF EXISTING BUILDINGS.


(N) anchor bolt,
(also see alt. detail)
(N) continuous steel angle
weld
(N) steel strap nailed into blocking
(N) plywood diaphragm overlay
(E) wood floor
(N) blocking
(E) wall
22° minimum
alternate anchorage detail for blind wall


(E) wall
(E) floor or roof
(N) blocking
(E) joist
(N) nails into blocking
(N) metal strap anchor lag bolted to new blocking
(N) through rod (welded to metal strap)


(E) wall
(N) blocking
(E) floor or roof
(E) joist
(N) metal strap anchor
(N) through rod


(E) wall
(E) diaphragm
(N) sleeve nut
(E) joist
(N) anchor bolted to joist
(N) torque nut
(N) anchor